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Rainfall Induced Infiltration Into Sewer Systems

Report To Congress

ACKNOWLEDGEMENTS

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RAINFALL INDUCED INFILTRATION INTO SEWER SYSTEMS

REPORT TO CONGRESS

August 1990

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This Report to Congress, required by Section 523 of the Federal Water Quality Act of 1987 (Public Law 100-4), presents the results of an Environmental Protection Agency (EPA) study of rainfall induced infiltration (RII) into municipal sanitary sewerage systems. The following are the objectives of the study:

- Study problems associated with RII.
- Study appropriate methods to control RII into municipal sanitary sewerage systems, including that of the East Bay Municipal Utility District, California.
- Develop recommendations on reasonable methods to reduce RII.

BACKGROUND

The Clean Water Act (CWA) of 1972 clearly established the intent of Congress to address problems associated with the entry of extraneous storm water and ground water (termed infiltration/inflow, or I/I) into sanitary sewer systems. The CWA mandated that all "excessive" I/I be removed from a sanitary sewer system as a condition for award of a construction grant for wastewater treatment facility improvements. "Excessive" flow was defined as that portion of the total extraneous flow that could be cost-effectively removed. That is, the cost to eliminate the excessive flow would be less than the cost to transport it in the sewer system and provide wastewater treatment.

Based on the requirements of the CWA, EPA developed guidelines for identifying extraneous flow, and specifically for determining what portion of the extraneous flow was excessive. A key concept in these guidelines was the distinction between "infiltration" and "inflow." In general, infiltration was used to describe the long term seepage of water into sewers through underground defects in the system. Such seepage was not considered to be directly related to recent storm events. Inflow was defined as water entering sewers through direct connections, such as cooling water from commercial and industrial buildings, cellar or yard drains, or roof downspouts connected to sanitary sewers.

In the years following the enactment of the 1972 law, communities throughout the country undertook sewer system rehabilitation programs to remove the flow that had been categorized as excessive. Flows were reduced in a number of such systems, while in others the anticipated flow decreases did not occur. One explanation of why these programs failed to achieve the expected results is that infiltration (which is generally difficult and expensive to correct) may have been incorrectly identified as inflow, resulting in an invalid or substantially overestimated assessment of the cost effectiveness of correction. Such situations can occur when extraneous flows enter the sewer system through traditional infiltration points, but produce a peak flow response similar to that of inflow.

In 1987, Congress asked EPA to investigate this problem. We conducted case studies in 10 cities or sewer districts. These studies attempted to gather information on: an appropriate definition of rainfall induced infiltration (RII); the characteristics of RII; the problems associated with RII; the pathways and entry points into the sewer system; and the major factors which influence the occurrence of RII. Data on methods to control or correct RII were obtained from the 10 case studies and augmented through a review of the pertinent literature.

FINDINGS AND CONCLUSIONS

The findings and conclusions of the study are grouped into two sections, corresponding to the first two study objectives: to assess the problems associated with RII, and to study methods of RII control.

Problem Assessment

The major findings and conclusions of the study with respect to the characteristics of, and problems associated with, RII are listed below:

- RII is a type of infiltration since it enters the sewer system through defects. However, its flow characteristics resemble those of inflow i.e., there is a rapid increase in flow which mirrors the rainfall event followed by a decrease as the rain stops.

- Because of its flow characteristics, RII has been misidentified as inflow in many cases. Consequently, rehabilitation programs have not achieved the anticipated reductions in extraneous flows.

- RII appears to represent a significant portion of the flow to sewage treatment plants during wet weather periods. In the 10 case studies the peak wet weather flow ranged from 3.5 to 20 times the average dry weather flow. The contribution from RII was estimated to be between 60-90 percent of the wet weather flows. The remainder is "traditional" groundwater infiltration and inflow.

- Collection and treatment systems typically do not have the capacity to handle peak wet weather flows. Peak flows, therefore, can cause backups into buildings, overflows and treatment system bypasses. Such occurrences are a hazard to public health or a violation of the municipality's discharge permit.

- o Sewer trenches act as collectors of rainfall percolating into the soil. The trenches channel the water, thus providing multiple opportunities for the water to seep into the collection system at defective points.

- o The shallow portions of a collection system (building laterals and their connections, sewer mains, manhole defects and foundation drains) are more vulnerable to RII. Interceptors, which are typically deeper, do not appear to be a significant entry point.

- o The extent of RII in a sanitary sewer system is related to design, construction, climate, geology and degree of maintenance.

RII Control Methods

RII control means the implementation of measures to reduce existing RII flows or limit future RII into a sewer system. RII control can be accomplished through various means, including physical rehabilitation of the sewer system, improved design standards and construction practices, preventive maintenance, and institutional and regulatory approaches. The major findings and conclusions of the study with respect to the various methods and approaches for RII control are:

- o Accurate field investigations and data analyses are important for developing an effective RII control program.

  The first step in developing an effective control program is to accurately identify and quantify RII in the sewer system, and to distinguish RII from other I/I components. The traditional I/I field data collection techniques commonly in use can be successfully used for RII investigation as long as the techniques are properly applied and the data correctly interpreted. For example, flow monitoring sites should not be influenced by severe pipe constrictions, and hydraulic conditions must be considered in interpreting flow monitoring results.

- o Many methods are available for rehabilitation of sewer systems to reduce RII.

  Pipeline rehabilitation methods include in-place techniques, such as grouting and lining, as well as replacement by excavation or trenchless installation methods. The suitability of different methods for correcting RII problems depends upon cost, extent of problem, and site-specific physical conditions, including the condition of the existing pipes.

Manhole rehabilitation techniques include both interior and exterior repair methods. Many of these methods are specifically designed to eliminate RII which seeps into pavement cracks and enters the sewer system through manhole frame and chimney defects.

- The traditional approach to determining the cost effectiveness of sewer system rehabilitation to reduce extraneous flows evaluates each inflow source or defective sewer component on an individual basis. This traditional approach can overestimate the amount of flow reduction achievable from rehabilitation because it fails to account for the migration of water to defects that are left unrepaired.

- A comprehensive program of sewer system rehabilitation that includes both the public and private portions of the system can be effective in reducing RII, although sometimes at considerable cost. If the private portion is not included, a significant portion of the RII may not be addressed. Water may also migrate to unrepaired defects in public portions of the system, thereby reducing the effectiveness of the rehabilitation effort.

RECOMMENDATIONS

- The specific analysis of RII should be included as part of overall I/I evaluations. Guidelines should be developed to ensure the proper application of field techniques and interpretation of data to identify and evaluate RII.

- The following considerations should be incorporated into the development of sewer system rehabilitation programs and evaluation of the cost effectiveness of rehabilitation:

  - Addressing entire areas of the sewer system versus repair of individual defects only.

  - Including both the public and private portions of the sewer system versus only the public portion.

- Long-term control of RII should be ensured through implementation of an effective preventive maintenance program that includes:

  - Periodic flow monitoring in the system to identify areas with increases in RII levels.

- A routine program of cleaning and root removal.

- A cyclic program of testing and inspection of the sewers throughout the system to identify the need for repairs and replacement.

o Sewer design standards should be modified to provide a cost-effective means to minimize future RII into new or rehabilitated sewers by controlling the development of extraneous water in sewer trenches.

o Effective sewer construction practices should be followed by:

- Rigorous construction inspection.

- Effective performance testing for public sewer mains as well as private laterals.

o The institutional and regulatory framework governing the construction and maintenance of house laterals (the connection between the house or building and the collector sewer in the street or other public right-of-way) should be re-examined. Possible options include:

- Shifting responsibility for construction and/or maintenance of house laterals from the home owners to the municipality.

- Municipal programs to help home owners pay for maintenance and repairs of house laterals.

- State or municipal ordinances, with appropriate enforcement provisions, governing inspection, testing and repair of house laterals.

- Public education programs to inform citizens of the importance of excluding extraneous flows from the municipal sanitary sewerage systems.

DEFINITION

A number of closely related phenomena are discussed throughout this report. For the convenience of the reader, short definitions of these phenomena and the acronyms used in the report are included below. Also included are schematic drawings and graphs to help the reader visualize these phenomena.

Infiltration

Water other than wastewater that enters a sewer system (including sewer service connections and foundation drains) from the ground through such means as defective pipes, pipe joints, connections, or manholes. Infiltration is typically not intentional and occurs by seepage through defects in the system. The contribution of foundation drains is considered as infiltration due to its rate and duration characteristics even though it is an intentional contribution to the system. Total infiltration is composed of Rainfall Induced Infiltration and Ground-Water Infiltration.

Rainfall Induced Infiltration (RII)

RII is a particular form of infiltration which behaves like and is sometimes confused with storm water inflow. RII generally occurs during and immediately after rainfall events and it is believed to be caused by the seepage of percolating rainwater into defective pipes (in many cases service connections or laterals) which lie near the ground surface. These circumstances cause a large portion of the rainfall to enter the system relatively quickly and the extraneous flow lasts only a short time after the rainfall episode is over. The combination of these factors causes RII to be of relatively short duration and high intensity as compared with typical infiltration which is generally constant in intensity and of longer duration.

Ground-Water Infiltration (GWI)

GWI results from the movement of ground water in the saturated zone into the sewerage system through defects in the components of the sewer system located below the water table. GWI is relatively constant and is generally not significantly affected by rainfall events (except where the ground-water is near the sewer pipe).

Inflow

Water other than wastewater that enters a sewer system (including sewer service connections) from sources such as roof leaders, cellar drains, yard drains, drains from springs and swampy areas, manhole covers, cross connections between storm sewers and sanitary sewers, catch basins, cooling towers, storm waters, surface runoff, street washwaters or drainage. Inflow is generally easier to locate and eliminate from the system than infiltration because it enters from specific points that can be identified and closed off.

DEFINITION

Storm Water Inflow (SWI)

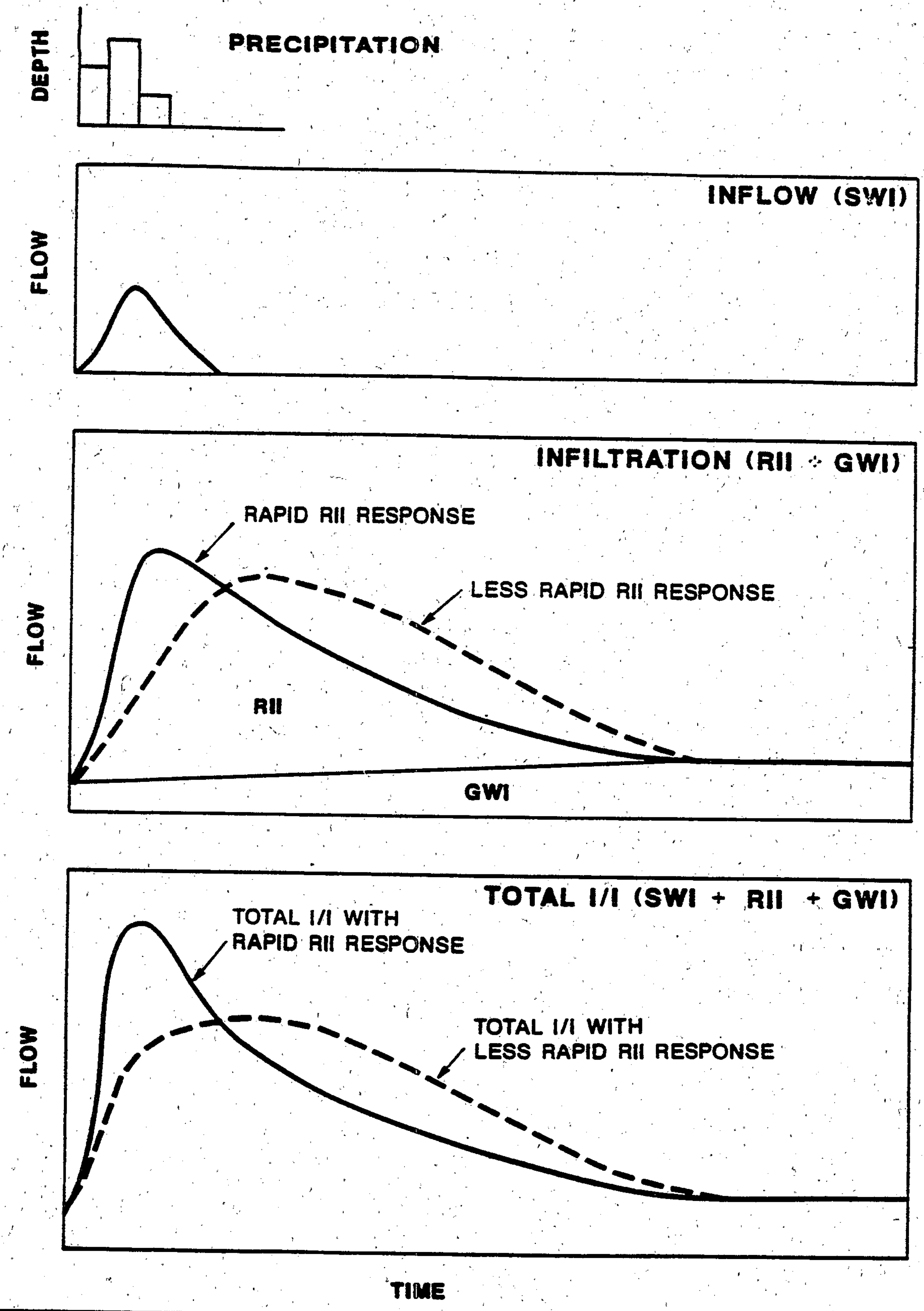
SWI is generally the result of intentional diversion of storm water into sanitary sewers. These connections are usually easy to identify and correct. The pattern that they follow is a prompt response which mirrors the rainfall event, followed by a quick decrease as the event stops. An example of SWI is roof downspouts which are connected to a sanitary sewer line.

Dry Weather Inflow (DWI)

DWI is the result of extraneous contributions to the flow of the sewer, which are not caused by rain. Some examples are water from street washing that enters manholes through the holes in the covers, cooling water for industrial and commercial applications, and some car washing activities.

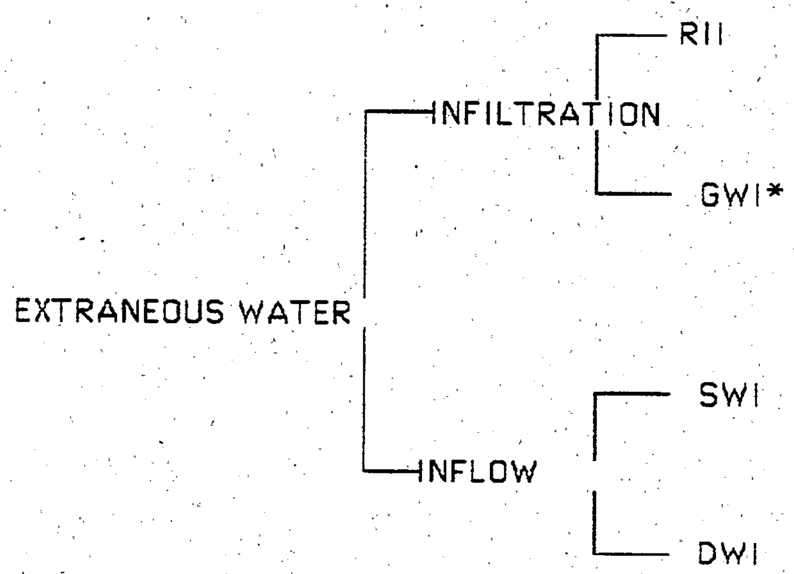
Infiltration/Inflow (I/I)

This is the combination of all the extraneous contributions to the sewer system. I/I is equal to RII + GWI + SWI + DWI (see graph).

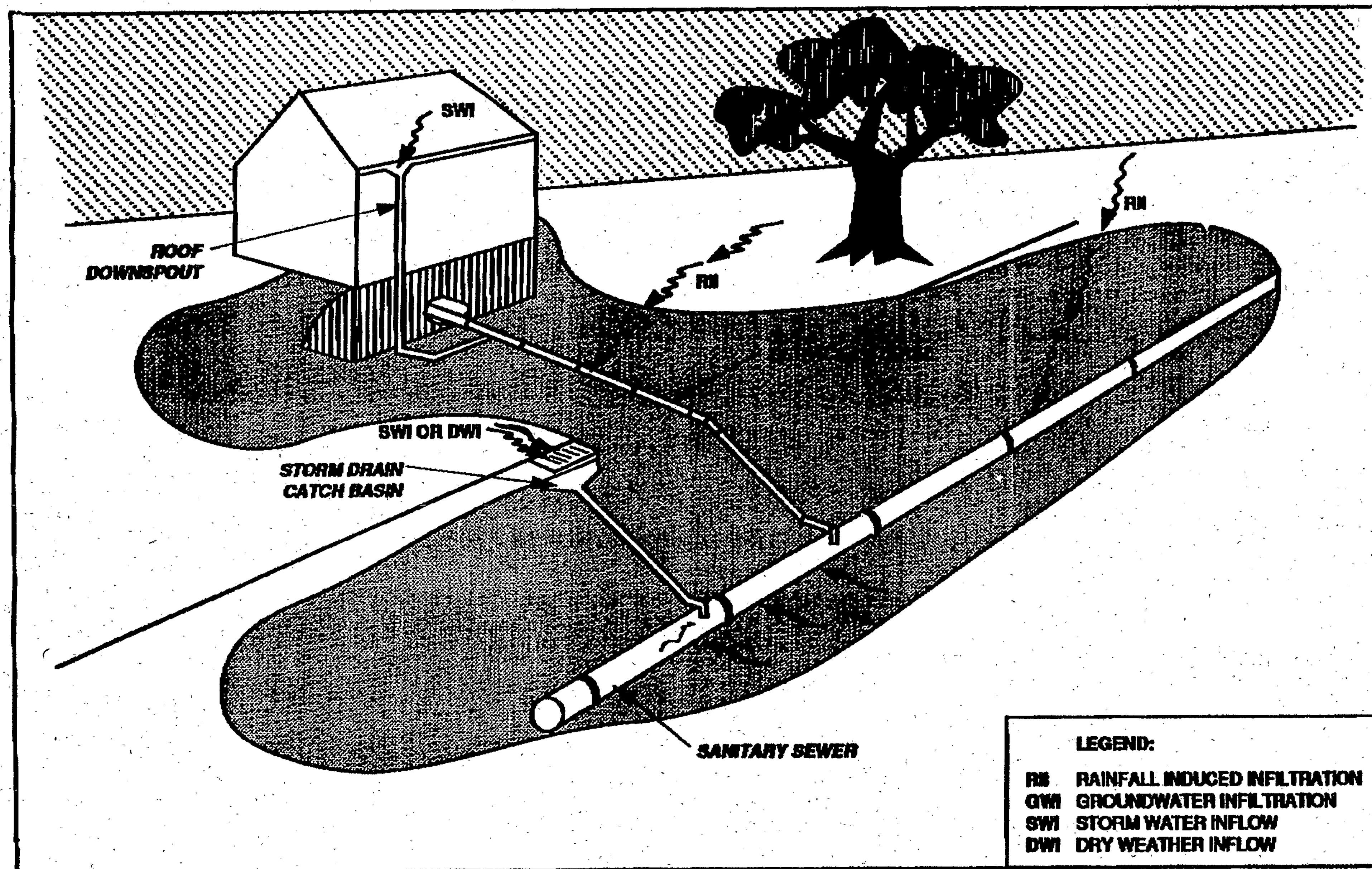


TYPICAL EXTRANEOUS FLOW HYDROGRAPHS

SCHEMATIC FOR SEVERAL TYPES OF EXTRANEOUS WATER INTO SEWERS



*INCLUDES FOUNDATION DRAINS



RAINFALL CAUSED
DRY WEATHER I/I

EXTRANEOUS FLOW COMPONENTS

CHAPTER 1

INTRODUCTION

STUDY AUTHORIZATION AND OBJECTIVES

Section 523 of the Federal Water Quality Act of 1987 (PL 100-4) requires that the U.S. Environmental Protection Agency (EPA) conduct a study and submit a report to Congress concerning rainfall induced infiltration (RII) into sanitary sewer systems. The specific requirements of Section 523 were to:

- o Study problems associated with RII.
- o Study appropriate methods to control RII into sanitary sewer systems, including that of the East Bay Municipal Utility District (EBMUD), California.
- o Develop recommendations on reasonable methods to reduce RII.

STUDY APPROACH

An approach was developed to accomplish the goals of the study, as follows:

- o Establish a definition of RII.
- o Identify sewer systems in the United States which experience RII, and document the characteristics of and problems associated with RII in those systems.
- o Conduct a literature search of applicable methods for controlling the entry of RII into sanitary sewer systems.
- o Conduct an evaluation of the costs of various approaches to control RII.
- o Develop recommendations on appropriate methods and approaches for RII control.

REPORT ORGANIZATION

The report is divided into several chapters and appendices. This chapter briefly describes the study objectives and approach. Chapter 2 presents an assessment of the RII problem, including the definition and characteristics of RII, a discussion of the problems associated with RII, and the presentation of ten case studies of sanitary sewer systems identified as experiencing RII. Chapter 3 discusses methods and approaches for controlling RII. The appendices contain more detailed descriptions of the case studies, further information on rehabilitation methods and design standards for RII control, and a detailed discussion of the RII cost evaluation conducted for this study.

CHAPTER 2

PROBLEM ASSESSMENT

This chapter discusses the characteristics of and problems associated with rainfall induced infiltration (RII) into sanitary sewer systems. Included are a definition of RII; a discussion of the typical problems associated with RII; a description of possible pathways by which rain can be rapidly transported from the ground surface to where it enters a sanitary sewer system; a discussion of the types of defects and connections through which RII may enter a sewer system; an assessment of the key factors which may be important for explaining the potential for RII occurrence in specific sewer systems; and a summary of RII case studies.

BACKGROUND

The entry of extraneous water into sanitary sewer systems has been recognized for many years as a significant problem in communities throughout the country. This extraneous water, termed infiltration and inflow (I/I), consists of groundwater and storm water which enter the sewer system through defects in pipes and manholes and through direct connections to the sewer system. When present in excessive amounts, I/I can cause wastewater overflows and bypasses from manholes and pump stations, bypassing and/or inadequate processing of wastewater at treatment plants, and flooding of building basements with wastewater.

The need to address excessive I/I was dictated in the Federal Water Pollution Control Act Amendments of 1972 (PL 92-500). Under this law, Congress mandated that all "excessive" I/I be removed from a sanitary sewer system before a construction grant for wastewater treatment facility improvements could be awarded. EPA has interpreted "Excessive" I/I as that portion of the total I/I which could be cost-effectively removed, i.e., the cost for removal would be less than the cost for transport and treatment of the "excessive" I/I flows.

In the years immediately following the enactment of the 1972 law, the EPA developed guidelines for conducting I/I cost-effectiveness analyses and sewer system evaluation surveys (SSESs) to identify excessive I/I (Appendix B). EPA regulations at 40 CFR Part 35 define the terms "infiltration" and "inflow" as follows:

> **Infiltration.** Water other than wastewater that enters a sewer system (including sewer service connections and foundation drains) from the ground through such means as defective pipes, pipe joints, connections, or manholes. Infiltration does not include, and is distinguished from, inflow.

> **Inflow.** Water other than wastewater that enters a sewer system (including sewer service connections) from such sources as, but not limited to, roof leaders, cellar drains, yard drains, area drains, drains from springs and swampy areas, manhole covers, cross connections between storm sewers and sanitary sewers, catch basins, cooling towers, storm waters, surface runoff, street wash waters, or drainage. Inflow does not include, and is distinguished from, infiltration.

In general, the understanding of infiltration was that it entered the sewer system <u>indirectly</u> via groundwater seepage into underground sewer defects, whereas inflow was rainfall runoff entering through <u>direct</u> connections. An exception to this generalization was later made when directly connected foundation drains were reclassified as infiltration rather than inflow, thus recognizing the sustained flow contribution of foundation drains in areas of high groundwater.

The EPA guidelines described procedures for separating and quantifying infiltration and inflow by use of flow data. Specifically, infiltration was calculated as the difference between total flow and estimated wastewater input on non-rainfall days. Inflow was calculated as the difference between the total flow during a large storm event and the total flow on the nearest non-rainfall day. Thus, in practice, the term "inflow" came to be synonymous with short-term, rain-induced I/I. The EPA guidelines acknowledged that both infiltration and inflow are affected by rainfall, but that it was not possible to precisely quantify infiltration and inflow in accordance with their literal definitions. As a result, it was concluded that the accuracy levels of the calculated values were adequate for estimating that portion of the I/I which might be considered excessive.

Subsequently, communities throughout the country conducted I/I analyses and SSESs using the EPA guidelines, and many undertook sewer system rehabilitation programs to remove the I/I that had been categorized as excessive. While I/I flows were reduced in a number of such systems, in others, the anticipated flow decreases did not occur. One possible explanation of why these programs failed is that infiltration may have been incorrectly identified as inflow. This can happen when water infiltrates into the sewer system through pipe and manhole defects, but produces a peak flow response similar to that of inflow from direct connections. Inflow connections can typically be eliminated at a lower cost (per unit of flow removed) than can defects in pipes and manholes. Therefore, if flows due to infiltration are incorrectly identified as being due to inflow, an invalid or substantially overestimated assessment of the cost effectiveness of I/I correction may result.

Problem Assessment

One wastewater system with extremely high rain induced extraneous flows is the East Bay Municipal Utility District (EBMUD) in California, which includes the City of Oakland and six adjacent communities. During large rainfall events, the EBMUD system can experience flows as high as twenty (20) times the average dry weather flow. As a result, peak flows exceed the conveyance capacity of the sewer system, causing overflows onto city streets and bypasses of untreated wastewater to San Francisco Bay.

To address these problems, EBMUD and its tributary communities undertook extensive studies to identify and quantify the rainfall induced extraneous flows in their sewer system. The goal of these studies was to develop a regional plan to eliminate peak flows that could cost-effectively be reduced, and then to adequately process the remaining volume of wet weather wastewater.

The comprehensive I/I study conducted by the EBMUD communities concluded that only a small fraction of the high peak flows occurring during rainfall events could be attributed to direct inflow. The majority of the rainfall induced flow was attributed to infiltration, and was called "rainfall dependent infiltration" in the EBMUD studies. Thus, EBMUD became the impetus for the study on rainfall induced infiltration called for under the 1987 Water Quality Act.

DEFINITION OF RII

For the purpose of this report, we have defined rainfall induced infiltration (RII) as follows:

> **Rainfall Induced Infiltration.** RII is a particular form of infiltration which behaves like and is sometimes confused with storm water inflow. RII generally occurs during and immediately after rainfall events and it is believed to be caused by the seepage of percolating rainwater into defective pipes (in many cases service connections or laterals) which lie near the ground surface. These circumstances cause a large portion of the rainfall to enter the system relatively quickly and the extraneous flow lasts only a short time after the rainfall episode is over. The combination of these factors causes RII to be of relatively short duration and high intensity as compared with typical infiltration which is generally constant in intensity and of longer duration.

Rainfall induced infiltration can be distinguished from "classical" infiltration because it results in a peak flow response in sanitary sewer systems which may be indistinguishable from that of direct storm water inflow. For the purposes of the discussion in this report, the long-term, sustained classical type of infiltration will be described by the term "groundwater infiltration" (GWI). "Storm water inflow" (SWI) will be used as the term for direct inflow as defined by EPA. Both GWI and RII are forms of infiltration, as described by the EPA definition, but differ primarily in their flow response.

The distinctions between SWI, GWI, and RII are illustrated by the hydrographs in Figure 2-1. As shown in the figure, SWI produces a rapid, peak flow response to rainfall which recedes quickly after the rainfall stops. Rainfall may also produce a net increase in the sustained GWI flow rate, as shown in the figure. RII response may be as rapid as that of SWI, or may include a delayed response which lags the peak rainfall intensity by several hours and then recedes slowly. In most sewer systems, the RII response is likely a continuum from a rapid peak flow to a more gradual, prolonged response similar to GWI. Therefore, the separation between the RII and GWI portions of the hydrograph may not be well-defined. RII becomes most significant when the type of flow response is more like inflow, i.e., it results in a rapid and high peak flow in the sanitary sewer system.

PROBLEMS ASSOCIATED WITH RII

The problems associated with RII are those due to the high peak flows which occur during and immediately following rainfall. Typical RII problems include wastewater overflows and bypasses from manholes and pump stations in the sewer system, and flooding of building basements. Wastewater backing up into homes or overflowing into city streets is a hazard to public health and, in most cases, is a clear violation of the discharge requirements of the sewerage agency. Additionally, wastewater bypassed to drainage channels may result in water quality degradation in downstream surface waters. If the flows reaching the wastewater treatment plant are much higher than the plant's capacity, deliberate bypassing may be necessary to avoid hydraulically overloading the plant. At very high plant flows, inadequate wastewater treatment and inability to meet discharge requirements may result. In all cases, excessive RII flows result in increased operation and maintenance costs for transport and treatment.

An ancillary problem associated with RII is that there is the potential for exfiltration of untreated sewage at these same pipe and manhole defects. This problem is especially likely to manifest itself when the sewer pipe is above the water table. In some cases, discharged sewage may cause ground-water contamination; in other cases it might be channelled by sewer trenches to potential points of direct human exposure.

The peak nature of flows due to RII, and the magnitude of these flows in some systems, means that wastewater collection, transport, and treatment facilities must be designed for capacities that greatly exceed normal peak dry weather flows. Thus, very large capital expenditures may be required to construct facilities that

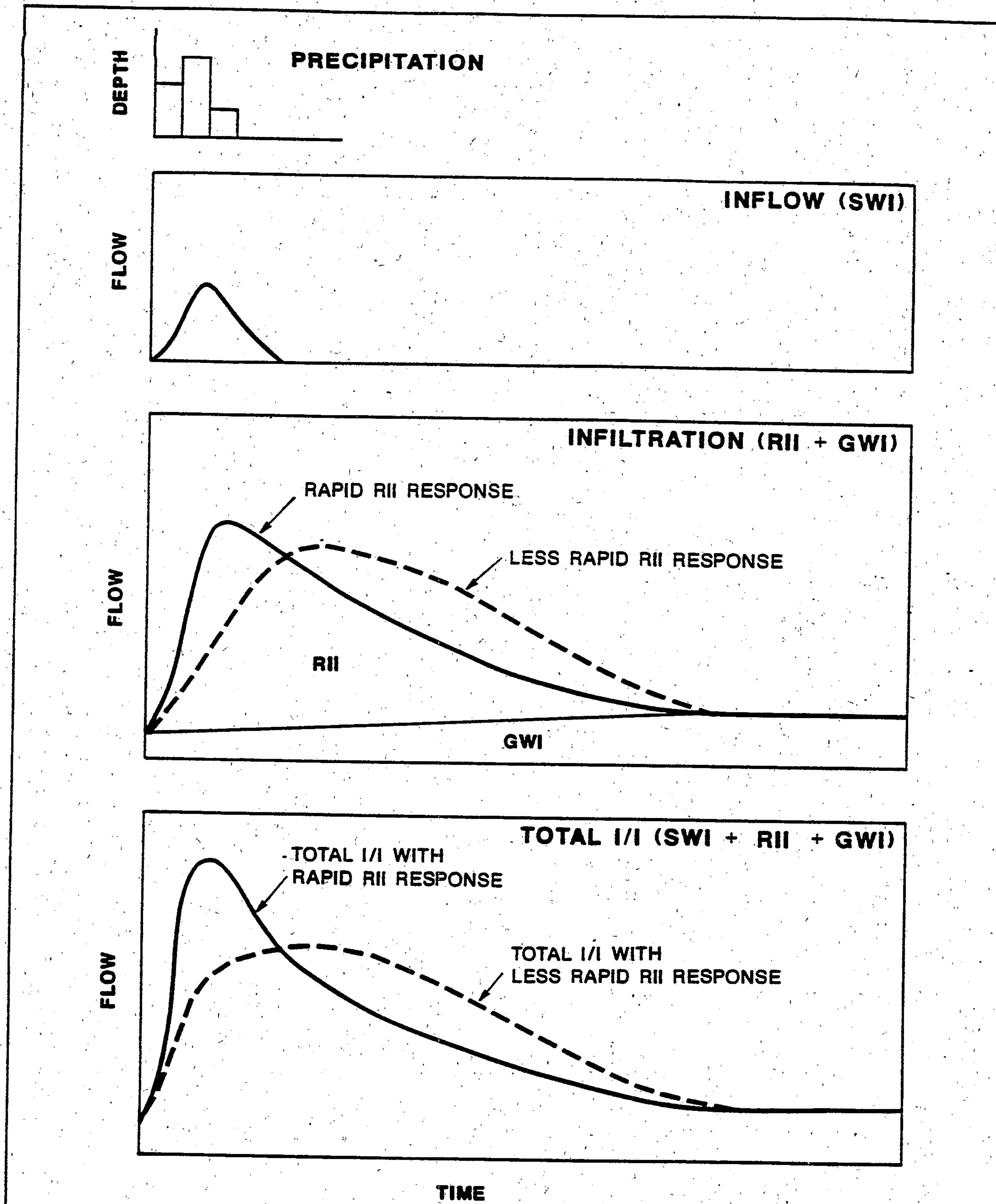


FIGURE 2-1

TYPICAL EXTRANEOUS FLOW HYDROGRAPHS

can handle the RII flows. Funding for such construction may be difficult, if not impossible, to obtain. Similarly, system capacity that might otherwise be available for future growth must be used for RII. In systems with severe capacity limitations and problems due to RII, building moratoriums may be necessary to restrict further increases in wastewater flows.

The alternative to providing excess system capacity to handle high RII flows is to reduce RII through sewer system rehabilitation. However, as will be discussed in more detail later in this report, achieving substantial RII flow reductions through rehabilitation can be very difficult and costly. Part of this problem is due to the fact that in many areas, a significant portion of RII may originate on private property (from building laterals and foundation drains). Many communities have invested considerable sums of money (both under local programs and with state and federal funding) in rehabilitation programs that have proven ineffective in reducing I/I flows. The failure of many of these programs has been due in part to the failure to properly identify RII as the major component of I/I, and to implement an adequate program for RII control.

As noted previously, RII has been identified as the primary cause of wet weather problems in the EBMUD wastewater system. During large storms, overflows occurred at over 175 locations within the community collection systems and about ten times each year from one or more of seven shoreline bypass points on the District's major interceptor sewer along San Francisco Bay. To eliminate these problems and comply with discharge requirements, EBMUD and its tributary communities have had to initiate a major program of sewer system rehabilitation and construction of facilities to handle wet weather flows, at a cost of over $600 million. The section on Case Studies presented later in this chapter describe the problems associated with RII in nine other sewer systems throughout the country.

POSSIBLE RII PATHWAYS AND FLOW RESPONSE

Storm water may reach sewer system openings through different pathways from the ground surface. The resulting RII flow response will vary depending upon the type and length of the pathway that the water follows. Factors such as the characteristics of the soils, geology, groundwater, topography, and trench backfill materials will influence the speed of the flow response. A very rapid response would be expected in situations in which the RII pathway is more like a direct channel to the sewer entry point. A slower response would be expected in cases where the permeable backfill material in the sewer trench acts as a drain for the water in the surrounding soil.

Some possible pathways scenarios which may help explain how and why RII occurs are described below. While these pathways present different conceptual models of RII, they are not necessarily mutually exclusive. RII in any particular sewer system may result from a combination of several different scenarios.

Soil Channels

Storm water may reach sewer defects through "channels" in the soil, as illustrated in Figure 2-2. The channels may be large enough to be called "holes," or may simply be continuous "macropores" from the ground surface to the system defect. The channels may be created by soil fauna such as worms or rodents, or by plant and tree roots. In clay soils with high shrink/swell capacities, surface cracks may open which extend to the sewer trench. With each rain, the percolating water may gradually enlarge the above described holes, macropores, or cracks.

It is also likely that soil channels within the pipe trench form via a similar erosion process by water which exfiltrates from leaky pipe joints and defects, and then infiltrates back into the system during low flow periods. Such joint-to-joint channels have been observed around excavated pipes, and also are evident where grout injected into a pipe joint reappears at another nearby joint.

Flow response in the sewer system due to water movement through soil channels would vary depending upon the size of the channels, the distance the water must travel to a sewer defect, and the surface characteristics of the ground. In particular, for a rapid response to occur (i.e., faster than the natural transmission rate of water through the soil), the soil channels or pores would have to be large enough to overcome capillarity (pore diameters of at least 3 to 4 mm). The length of the soil channel (distance from the ground surface to the RII entry point) would also impact the speed of the RII response, with shorter channels, such as those to shallower sewers, producing faster response times. Where the surface characteristics are such that the ground over the channel forms a natural depression for surface runoff collection, the soil channels would act like direct inflow connections, conveying surface water rapidly to defects in the pipe.

Shallow Impermeable Strata

Where a shallow, relatively impermeable soil layer or bedrock exists, rainfall percolating into the soil may create a perched water table, as shown in Figure 2-3, and may be carried rapidly to sewer trenches as the groundwater level rises in response to rainfall. RII response under this scenario may vary from rapid (i.e., similar to SWI) to gradual (i.e., similar to GWI), depending upon the depth and permeability of the overlying soil, the slope

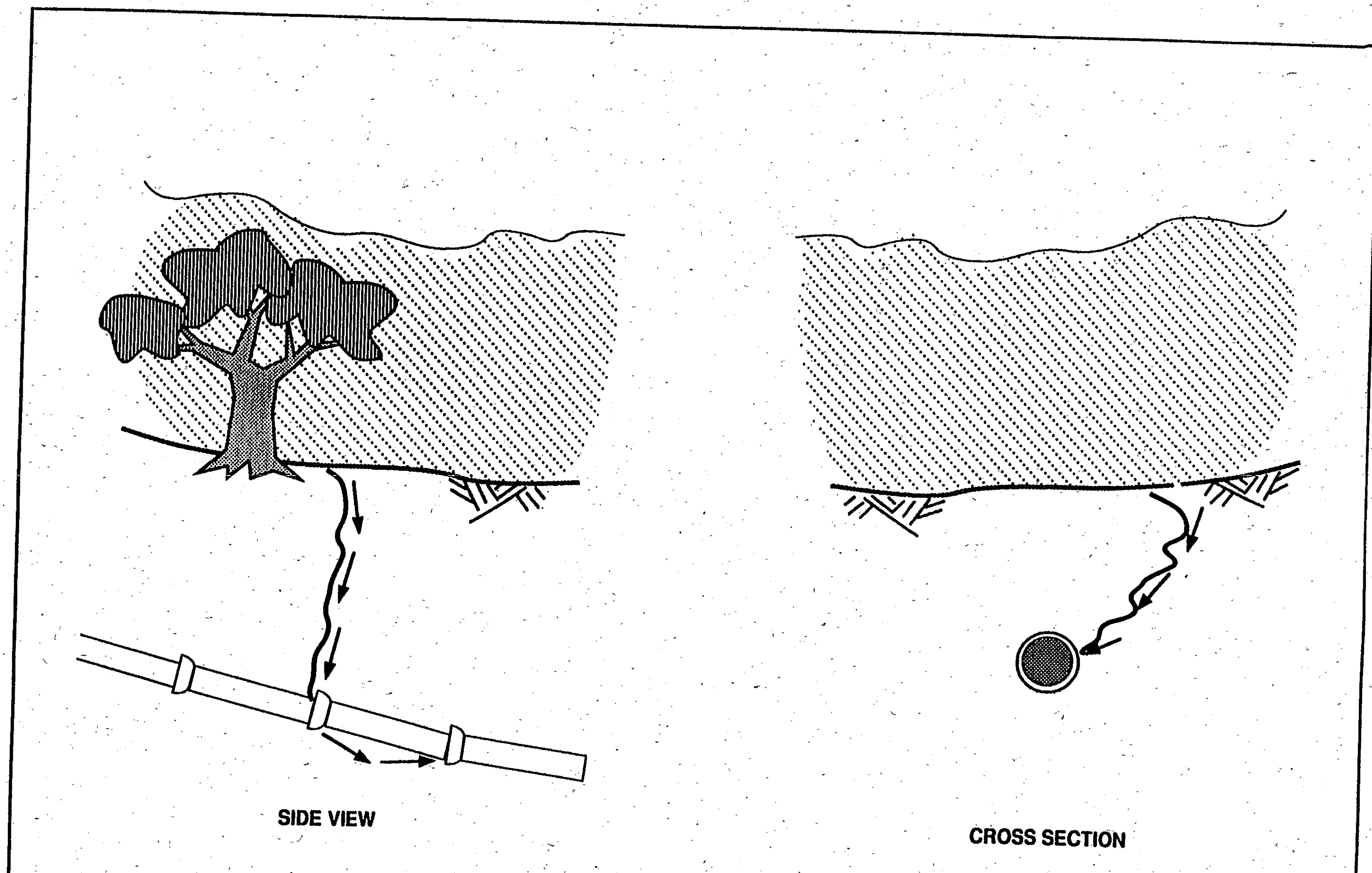


FIGURE 2-2

SOIL CHANNELS

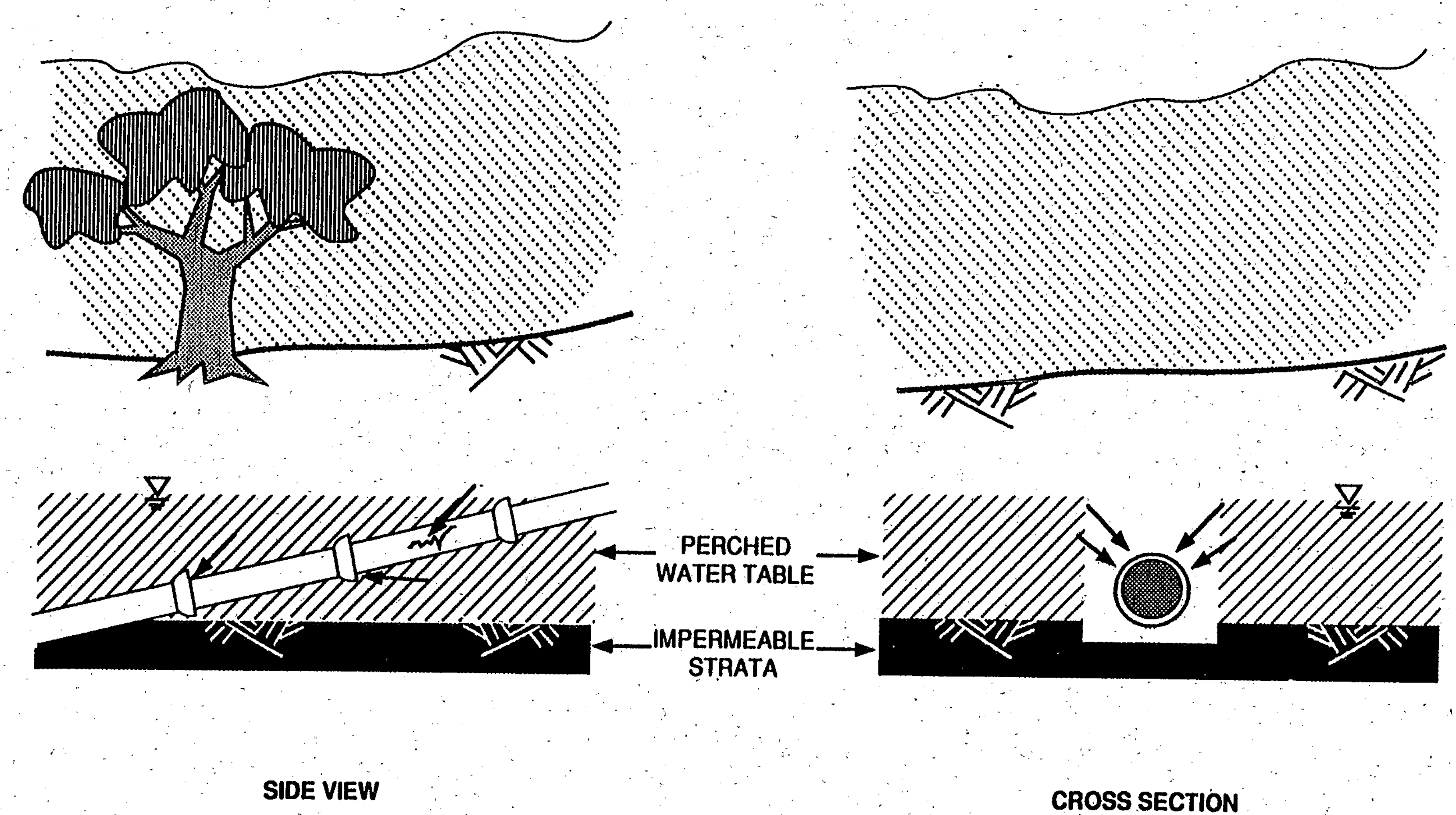


FIGURE 2-3

SHALLOW IMPERMEABLE STRATA

of the impermeable strata, and/or the extent to which the sewer trenches penetrate the impermeable material.

French Drain Effect

In some situations, a sewer trench may act like a French drain, an underground passage for water constructed of material that is "looser" (more permeable) than the surrounding soil. This condition would occur where the sewer trench bedding and backfill is composed of granular material (sand and/or gravel). As illustrated in Figure 2-4, the sewer trench would thus provide a conduit for water from the surrounding soil. If the surrounding soil becomes saturated because of rainfall, the sewer trench may drain the water, resulting in a rise in the transient water level in the trench. As the static water pressure over the pipe increases, the rate of RII into pipe defects will also increase. The RII response will typically be more gradual than that of SWI or "rapid" RII from soil channels. The French drain effect in a sewer trench may be accentuated by other pipe trenches crossing or intersecting the sewer trench.

Entry from Ground Surface into Sewer Trench Backfill

If trench backfill material is more permeable than the surrounding soil and extends to the ground surface, it may provide an area for rainwater on the ground surface to more easily infiltrate the trench, as illustrated in Figure 2-5. Any network of interconnected utility trenches can convey the water to the sanitary sewer trenches, typically the deepest utility, and to defects in the sewers. The RII flow response under this scenario would depend upon the runoff characteristics of the surface, surface topography, adequacy of existing storm drainage facilities, extent of the underlying trench network, depth of pipes, and type of trench backfill materials. Where slopes are steep and trenches are located in natural depressions (as is common for sewer trenches), RII flow response in the system could be rapid. In other situations, the response time could be more gradual.

A similar phenomenon may occur in cases where the sanitary sewer pipe parallels or crosses under surface drainage ditches. Storm water quickly collects and fills the ditches and infiltrates downward to the sewer pipe. In the extreme case, the sewer pipe may be installed directly under the entire length of a drainage ditch, resulting in rapid infiltration into the backfilled trench.

Storm Drain Exfiltration

Where sanitary sewer mains or laterals parallel or cross under storm drain trenches, water may exfiltrate from leaky storm sewers or storm laterals and then infiltrate into the sanitary sewer pipe,

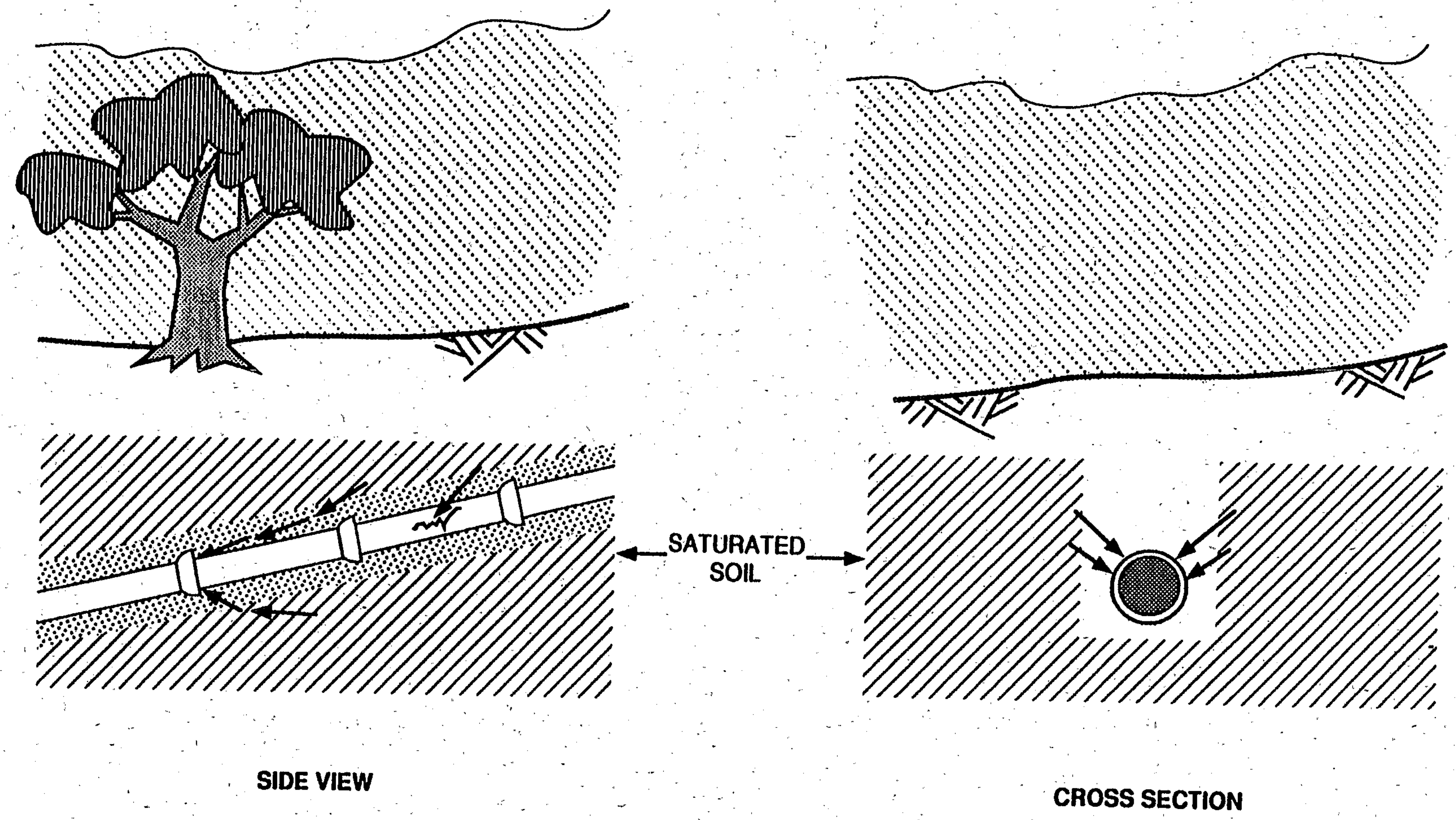


FIGURE 2-4

FRENCH DRAIN EFFECT

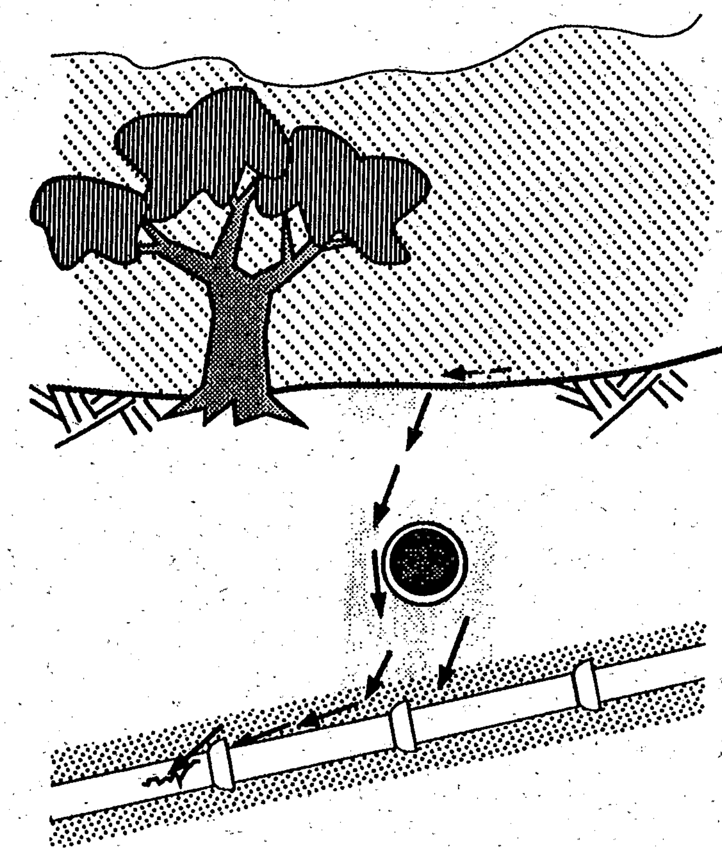


FIGURE 2-5

GROUND SURFACE TO INTERCONNECTED UTILITY TRENCHES

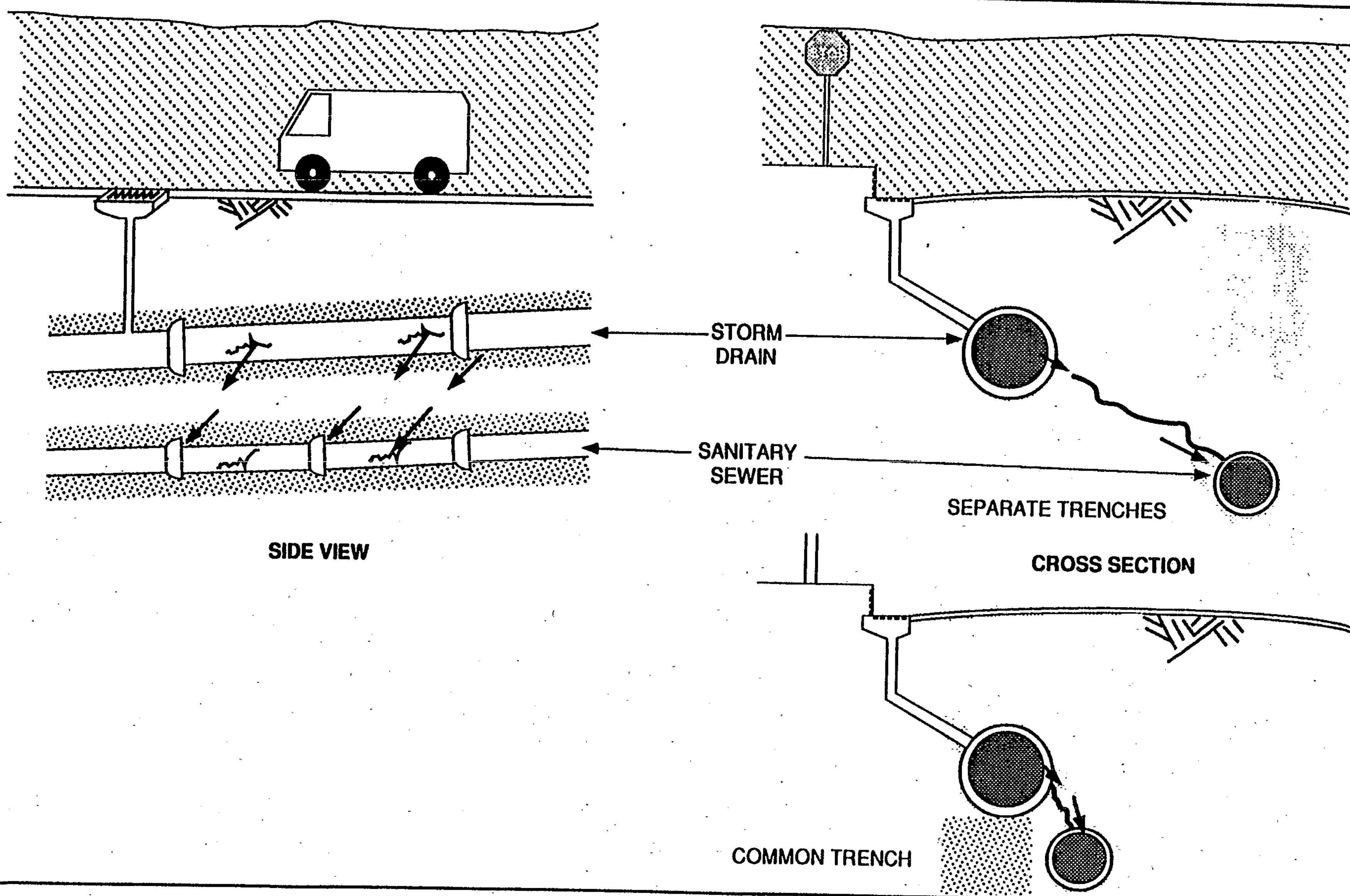


FIGURE 2-6

STORM DRAIN EXFILTRATION

as shown in Figure 2-6. Channels through the soil will gradually form between defects (exfiltration points) in the storm drain and defects (RII entry points) in the sanitary sewer. Since the stormwater would initially be conveyed very quickly from the surface to the exfiltration points in a storm drain, the RII response could be fairly rapid. In the extreme case, the storm drain or lateral may be installed in a common trench with the sanitary sewer pipe and backfilled with permeable material, resulting in a very short indirect cross-connection between the two pipes.

Subsurface Seepage

When streets are flooded during rainfall, water can seep into cracks in the pavement and travel laterally underneath the pavement to the upper portions of manholes, as shown in Figure 2-7. The water can enter the manholes through defects, typically between the manhole frame and chimney. Subsidence of trench backfill materials may cause channels to form between the pavement and street subbase. The street subbase, which is typically highly permeable material, could also function as a horizontal lens to direct the flow of water. Channels between the pavement cracks and manholes would gradually form through erosion. The RII flow response would be fairly rapid because the defects are located close to the ground surface, and horizontal water movement is promoted by street subbase material or channels. This pathway appears to be more common where freeze/thaw cycles occur in cold climates; both the cracks in the pavement and the openings between the manhole frame and chimney may be caused by such freezing and thawing of the ground.

Foundation Drains

Where foundation drains are used to lower the permanent or seasonal groundwater level from around building foundations, direct connections of the drains to the sewer system may exist, as illustrated in Figure 2-8. The foundation drains may contribute GWI during non-rainfall periods, but flow response may increase significantly during periods of rainfall. The magnitude and speed of the response would depend on lot slope, direction of surface drainage in relation to the building, location of downspout discharges, and permeability of the backfill materials next to the basement walls and drains.

Lee and Molzahn utilized a computer groundwater model to simulate the flow response in foundation drains from rainfall. The model demonstrated that foundation drains could produce a peak flow response that correlated more to total storm rainfall volume than to rainfall intensity. Rainfall simulation and wet weather flow measurements for foundation drains from other studies also

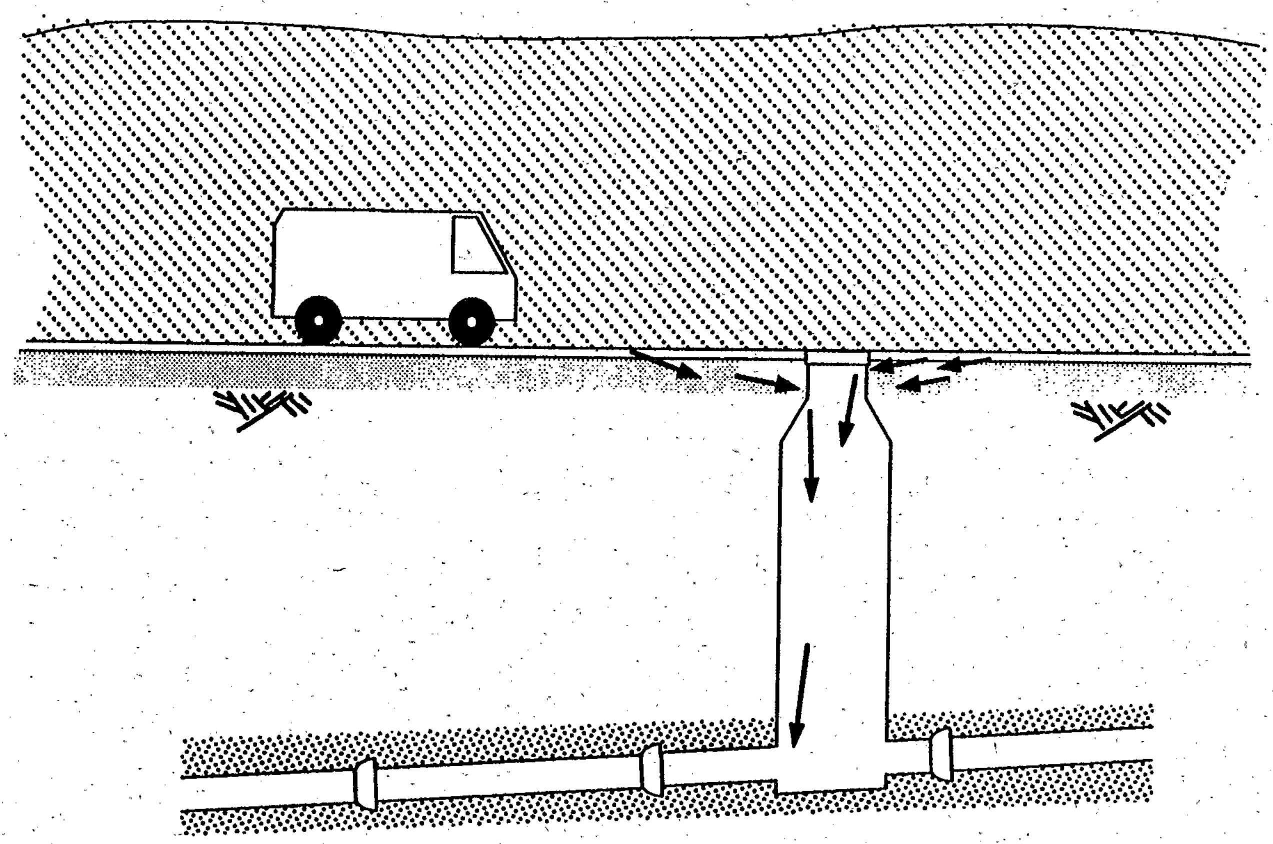


FIGURE 2-7

SEEPAGE THROUGH PAVEMENT CRACKS

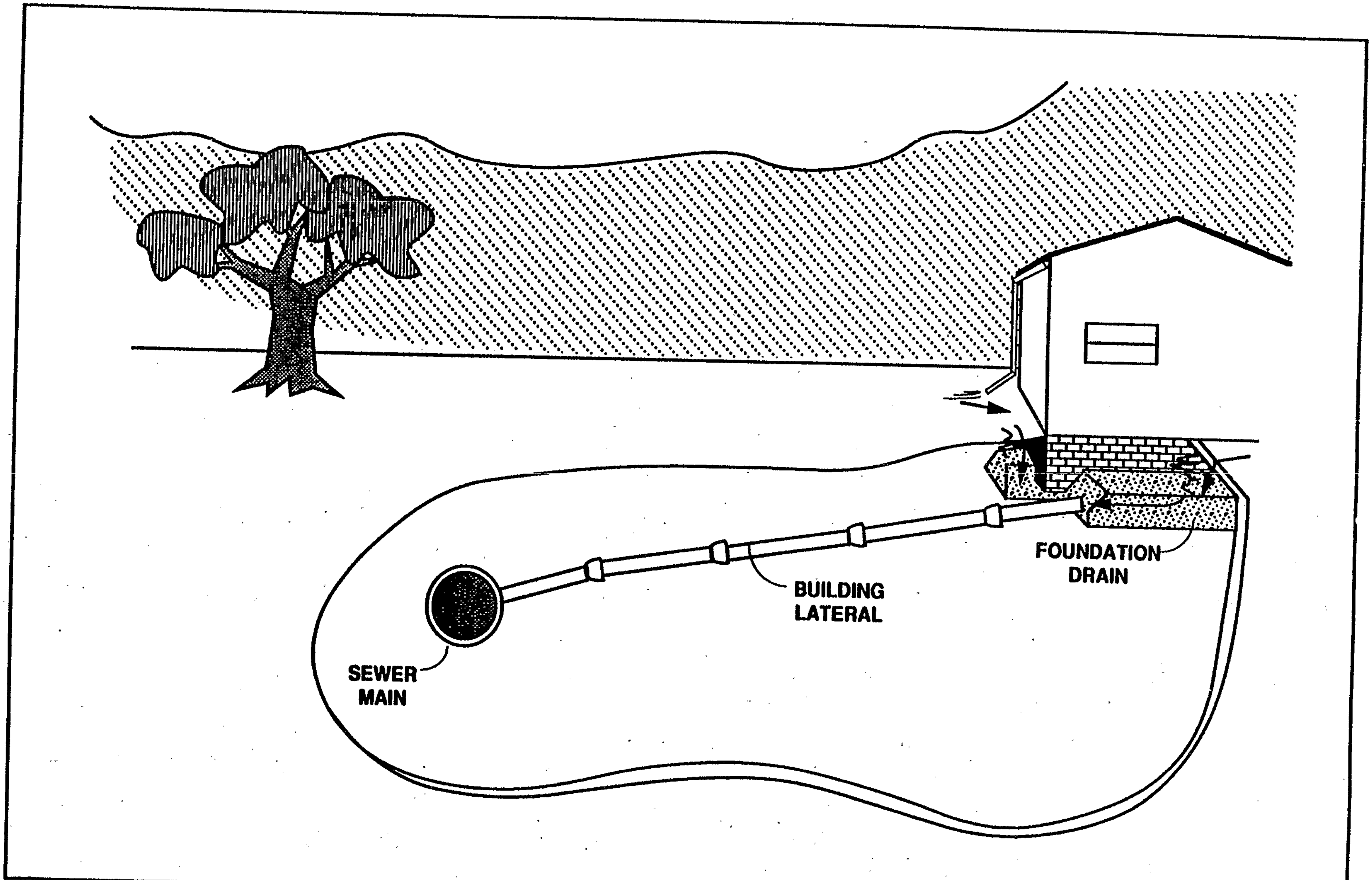


FIGURE 2-8

FOUNDATION DRAINS

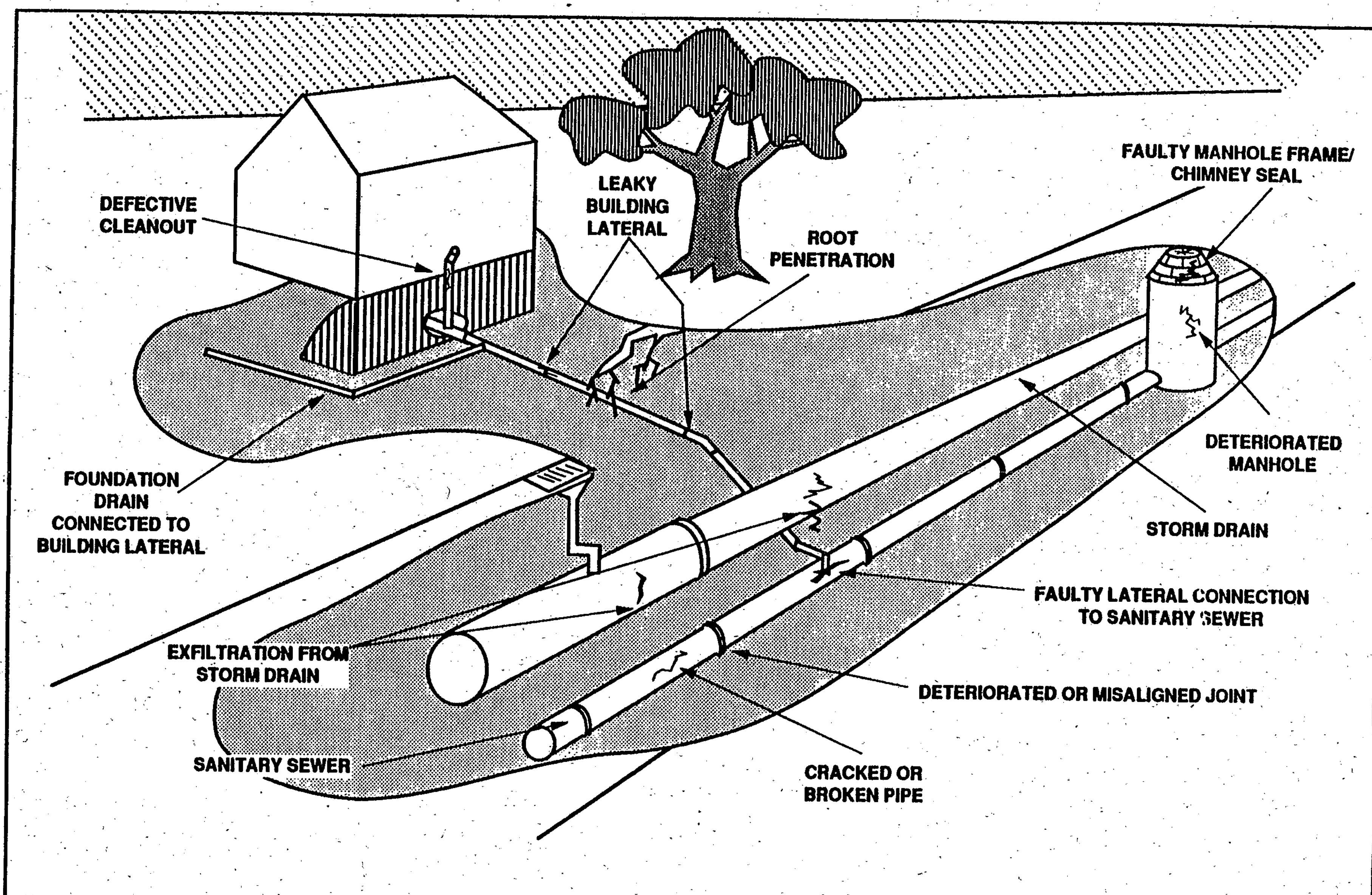


FIGURE 2-9

TYPICAL RII ENTRY POINTS

indicated that foundation drains can produce a peak flow response within one hour of rainfall.

ENTRY POINTS OF RII INTO SANITARY SEWER SYSTEMS

Extraneous water enters a sanitary sewer system through various types of openings. Infiltration entry points include defects in pipes and service laterals (cracks, holes, open or offset joints, defective pipe connections, etc.) and similar defects in other structures such as manholes and cleanouts. Foundation drain connections to sanitary sewer building laterals are also defined as infiltration entry points. Infiltration entry points are RII entry points whenever rainfall produces a significant, short-term increase in the flow of extraneous water. The various types of RII entry points are illustrated in Figure 2-9.

RII should not exist in a water-tight sewer system, i.e., a system where there are no openings for extraneous water to enter. No sewer system is expected to be completely water-tight; even new systems today are designed with a minimal allowance for infiltration. However, many systems, both old and new, have developed numerous defects which allow excessive amounts of extraneous water to enter. Typical RII entry points are described below.

Pipe Defects

Sewer systems installed in this country prior to about 1960 often have numerous defects. These defects are due to both poor construction practices and the materials that were used for construction. The short pipe lengths (two to three feet) installed in most older sewers resulted in many joints in the sewers. Specific problems have resulted because of:

- Low tensile strength of the pipe.
- High porosity of pipe materials.
- Hydrogen sulfide corrosion damage to concrete pipes.
- Cracking around the pipe bells due to the joint rigidity.
- Deterioration of the joint materials.

Better quality pipe and joint materials have come into widespread use since the 1960s. These include less porous, higher strength pipe, which is installed with flexible joints, as well as flexible pipe materials which come in longer pipe lengths (hence fewer joints). Use of low pressure air testing for determining the acceptability of newly constructed sewers has accelerated the transition to use of better pipe materials and has helped improve the quality of sewer construction.

Poor construction practices, both in old and new pipe installations, have also contributed to pipe defects. These include:

- Inadequate pipe bedding and poor backfill material compaction.
- Damage caused during construction of crossing utilities.
- Service lateral "hammer tap" connections.
- Unused, unplugged wye connections installed with the original sewer main.

In addition, external forces such as traffic loads, ground movement, and root intrusion also generate defects that become points of entry for RII.

Service laterals typically suffer from the same types of defects as sewer mains. However, the problems may be accentuated by the fact that the laterals are typically shallower, have shorter pipe lengths, are more subject to root intrusion, and are generally installed by less experienced contractors and subject to minimal testing or inspection. Often, laterals are broken into for cleaning purposes and not properly repaired or backfilled. Weak spots in laterals typically occur under the curb line, at the bend or vertical drop down to the sewer main connection, and at the sewer main connection. Service laterals may comprise more than half of the total pipe footage in a sewer system, hence may be significant contributors to RII.

Manhole Defects

Defects in manholes occur in the walls and joints, at the connections to the sewer pipes, and underneath the manhole frame. The joint between the manhole frame and chimney (corbel) may also be an entry point of RII when the frame is displaced or the joint seal is deteriorated, broken, or improperly installed. As with sewer pipes, manhole defects may also be created by external forces such as traffic loads, frost heave, and/or root intrusion.

Foundation Drains

Foundation or footing drains connected to building laterals are direct entry points for infiltration. Foundation drains are designed to drain the groundwater from around a building or house foundation to prevent seepage into the basement. The foundation drain may discharge by gravity or through a sump pump to the lateral. In some buildings without foundation drains, water seeping into the basement may be collected by the basement drain and similarly discharged to the sanitary sewer lateral.

FACTORS AFFECTING RII

In any particular sewer system, a variety of factors may influence how RII occurs and the magnitude and type of RII flow response. These factors relate to the construction and maintenance of the sewer system and the natural characteristics of the service area. Sewer system construction and maintenance affect the number, size, type, and location of openings through which RII can enter the system and the pathways by which water reaches the RII entry points. Natural characteristics of the service area primarily influence the pathways by which the rain water reaches the sewer system and the characteristics of the flow response pattern. Each of these various factors is discussed briefly in the following paragraphs.

System Age and Construction

Age is often an indicator of the type of sewer system construction and the types, severity, and relative number of defects that can be expected. As discussed previously, older systems, particularly those constructed before the 1960's, are often characterized by widespread defects due to the poor quality of the pipe and joint materials and methods used at the time of construction. These systems can be expected to contribute more RII than comparable newer systems under similar conditions of rainfall, soils, groundwater, etc. RII can also be expected to be higher in systems known to contain common trench storm drain and sanitary sewer installations.

Construction of houses with foundation drains connected to the sanitary sewer system was common in many areas during certain time periods. Relatively greater RII contributions from foundation drains would be expected in areas developed during these periods than in areas developed after direct foundation drain connections were prohibited.

Density

The magnitude of RII may be directly related to the amount of pipe within an area. Areas with denser development have more sewer main and lateral pipe footage, with a correspondingly greater number of potential RII points of entry. Hence, higher RII rates might be expected in areas with denser development.

Sewer Depth

The depth of sewers and laterals may influence the amount of RII and the speed in which it enters the sewer system. Where soil channeling or permeable trench backfill material extending to the ground surface are the pathways of RII entry into the system, shallower pipes can be expected to exhibit a more rapid RII response.

Groundwater

In areas with high groundwater, an increase in groundwater level due to rainfall may increase the submergence of the sewer. The greater hydrostatic pressure on the pipe may result in significantly higher rates of groundwater infiltration into the sewer, which in such cases could realistically be classified as RII.

Soils and Geology

The characteristics of the soils and geology of a service area will affect the rates of rainfall infiltration and percolation, and the occurrence of saturated soil zones. Permeable soils such as sands can transmit water rapidly; clay soils with large shrink-swell capacities can develop large channels. Hydraulically restrictive horizons or bedrock at or above the sewer trench bottom can create perched water table conditions during rainfall which greatly enhance water transfer to sewer defects. In soils subject to differential settlement, such as fills and bay muds, or in areas subject to earth movement from seismic activity, a greater number of pipe defects may develop, subsequently increasing the amount of RII which can enter the system.

Topography

Both water movement through the soil mantle and sewer flow rates are affected by topography. Sloped bedrock or impermeable soil layers will tend to cause perched groundwater to drain to sewer trenches. Sewers constructed on steep slopes carry flows more rapidly, resulting in higher peak flows in the system. These higher peaks may cause surcharging and overflows downstream in flatter portions of the system. Sewers and laterals constructed on steep slopes may be subject to earth movement, causing joint separation and other damage to the pipes. Topographic factors may also result in depressions or low areas over sewers, as well as close proximity of storm drains and drainage channels to sanitary sewers, a situation which can increase RII due to storm drainage exfiltration.

Roots

Root intrusion is a major cause of pipe defects in many areas. Roots enter sewer pipes through very small cracks and openings, enlarging these defects as root growth continues. Particularly in residential areas, private service laterals are often subject to root intrusion from plants and trees; trees lining the street may result in root penetration into both laterals and sewer mains. Root growth may also create channeling effects in the soil.

Rainfall Patterns

The magnitude and pattern of rainfall impacts the volume of RII and the type of response. In most systems that experience RII, it has been found that extended periods of rain produce larger volumes and higher peak RII flows than do isolated, short duration, high intensity storms. Highly seasonal rainfall patterns (i.e., prolonged periods without rain), as occur in the far western portions of the country, may create conditions that are more conducive to RII, e.g., creation of soil cracks and channels from the drying out of the soil during the prolonged dry season.

Cold Weather

Cold climate areas with substantial snowfall during winter experience higher RII flows when rainfall and snowmelt occur simultaneously. Peak flow patterns may also be produced by snowmelt alone. Frost heave may damage street pavements and manholes, creating openings for the rainfall and/or snowmelt to seep underneath the pavement and enter manholes below the ground surface.

Maintenance Practices

The number of new sewer defects through which RII may enter a system can be minimized by an effective preventive maintenance program. A system that has undergone routine preventive maintenance throughout its lifetime would be expected to contribute less RII than a system which lacks such a maintenance program. In general, very few sewer systems have been adequately maintained.

Typically, private building laterals are the most poorly maintained components of a sewer system. This situation is compounded by the fact that laterals are generally of originally poor construction. Most laterals have never been inspected, repaired, or replaced since original construction.

Ordinance Enforcement

Sewer ordinances may provide the institutional means for agencies to ensure the proper installation and maintenance of the private portions of their sewer systems. Examples are requirements for lateral installation by a licensed plumber or contractor, inspection prior to backfilling, requiring that connections to the sewer main be properly constructed and that abandoned or unused service laterals be plugged to prevent entry of extraneous water, and prohibitions against the direct connection of foundation drains to the sanitary sewer system. Where ordinances are not strictly enforced, RII can be expected to be greater due to illegal connections or major defects in service laterals left undetected and unrepaired.

CASE STUDIES

Ten case studies were documented for this study, including that of EBMUD and nine other systems that were selected through a candidate system search. Candidate systems were identified through contacts with EPA regional offices, regulatory agencies of each state, and major consulting engineering firms throughout the country. A list of approximately 350 possible candidate systems was initially compiled. After screening of preliminary information, over 65 telephone contacts were made to ascertain the likelihood of RII occurrence in candidate sewer systems and to determine what documentation was available. The general characteristics of RII, as defined under this study, were described, and each contacted agency was questioned as to the relative magnitude of peak wet weather flows, the known or likely sources of RII, and the availability of data from past studies. In general, most of the agencies contacted responded affirmatively when asked if it appeared that they had RII in their sanitary sewer systems.

Reports from approximately 40 systems were received. Most documented I/I analyses and SSESs completed in the late 1970s and early 1980s under various EPA projects. Therefore, the study methodologies and analyses employed largely conformed to EPA guidelines which were in effect during that period for identifying "excessive" I/I. The reports received were fairly representative of I/I studies completed over the past 15 years. The best candidates for case studies were considered to be those agencies which had documented potential pathways and entry points of RII, or could with reasonable certainty be assumed to have RII because of high peak flows with little or no known sources of direct inflow. However, only a very few had specifically addressed or attempted to quantify RII, or initiated programs designed solely to control RII.

Based on contacts made and documentation received, nine candidates for RII case studies were identified, as listed below:

- City of Springfield, Oregon
- Milwaukee Metropolitan Sewerage District, Wisconsin
- Northeast Ohio Regional Sewer District, Ohio
- City of Baton Rouge, Louisiana
- City of Springfield, Missouri

- North and South Shenango Joint Municipal Authority, Pennsylvania
- City of Ames, Iowa
- City of Coos Bay, Oregon
- City of Tulsa, Oklahoma

Further information was obtained from site visits to the first four systems and through written and telephone contacts with the others.

Detailed discussions of these case studies, including EBMUD, are included in Appendix C. Brief descriptions of the findings of each with respect to RII are presented below and summarized in Table 2-1.

East Bay Municipal Utility District, California

The EBMUD wastewater service area is located on the eastern shore of San Francisco Bay, and includes seven community wastewater collection agencies. EBMUD operates the interceptor system and treatment facilities which transport and treat the wastewater generated from the seven communities. The collection systems, which include about 1,500 miles of sewer main, are owned and operated by the individual communities.

The community collection systems, as well as the EBMUD interceptor and treatment facilities, do not have adequate capacity to handle peak Flows which occur during wet weather. As a result, overflows onto city streets and bypasses to local watercourses have occurred within the community systems and at seven locations along the EBMUD interceptor.

Findings documented from field investigations were:

- High peak flows occurred in response to rainfall. The ratio of peak wet weather flow (PWWF) to average dry weather flow (ADWF) was estimated to be about 20 to 1 for a five-year design storm.

- Identified direct inflow (i.e., SWI) accounted for less than five percent of the total rain induced extraneous flows.

- From smoke testing programs, numerous pipe defects were detected in building laterals.

- Numerous defects were observed in sewer mains and laterals through TV inspection programs.

- Very few direct storm drain/sanitary sewer interconnections were found. Most of the potential interconnections detected through smoke testing were found to be indirect (i.e., through pipe defects).

- o Laterals exhibited peak flow responses to actual rainfall and to simulated rainfall tests.

- o Most laterals given air or water leakage tests failed such tests.

The high rain induced flow response, the absence of significant direct inflow connections to account for any substantial portion of the peak flows, and the prevalence of defects in sewer mains and laterals indicated that RII is a significant component of peak wet weather flows in the EBMUD sanitary sewer system. The key factors affecting RII entry points appear to be the age and condition of the sewer system and the relatively high density of sewers and laterals. The poor condition of the pipes is primarily due to age and lack of maintenance, but is also affected by physical factors such as earth movement due to seismic activity. Other factors which contribute to the very rapid, high peak flows are the shallow depth of mains and laterals, clay soils, and steep slopes which characterize the service area.

The EBMUD communities have initiated a 20-year program to eliminate overflows and reduce RII in the sanitary sewer system. The recommended program consists of "comprehensive" rehabilitation (including sewer mains and the entire portion of building laterals) in approximately one-half of the subbasins in the system, coupled with construction of relief sewers to transport the excess flows not removed by rehabilitation. Rehabilitation work conducted during the initial phases of the program has consisted primarily of slip-lining and replacement of sewer mains and the portion of the building laterals within the public right-of way (lower laterals). One of the EBMUD communities has included the private (upper) laterals in the public construction project, and other communities are considering this approach for subsequent projects, as well as other options for implementing private lateral rehabilitation. Analyses of the flow reductions achieved through the initial rehabilitation projects are not yet complete.

City of Springfield, Oregon

Springfield is located in central western Oregon at the confluence of the McKenzie and Willamette Rivers. The City's sanitary sewer system is tributary to a regional wastewater treatment plant (WWTP) constructed in 1984, which serves the Cities of Eugene and Springfield. The Springfield sewer system serves a population of about 40,000 and includes approximately 165 miles of sanitary sewer mains. Problems in Springfield caused by rain induced flows have been reported to include system surcharging, overflows, and bypassing of partially treated wastewater from the former Springfield treatment plant (almost continuously during the months of December and January).

Findings documented from field investigations were:

- o High flows occurred in response to rainfall. The ratio of PWWF to ADWF was projected to be about 11 to 1.
- o Identified direct inflow accounted for less than 20 percent of the projected rain induced extraneous flow.
- o Numerous pipe defects were detected in sewer mains and building laterals from smoke testing studies.
- o TV inspection detected numerous defects in sewer mains.
- o Dye flooding tests confirmed that over 90 percent of the potential storm drain/sanitary sewer connections identified by smoke testing were through defects in the sewers rather than direct connections.

The high rain induced flow response, the fact that direct inflow connections accounted for less than 20 percent of the peak rain induced extraneous flows, and the prevalence of defects in sewer mains and laterals indicated that RII is a significant component of peak wet weather flows in the Springfield sanitary sewer system. The key factors affecting RII appear to be the condition of the sewer mains and laterals, groundwater conditions, and the high seasonal rainfall in the service area.

The City has conducted rehabilitation of the sewer mains and lower laterals in four areas of the system, utilizing primarily grouting and replacement. Rehabilitation of private laterals has also been done in several small special project areas. Analyses of the flow reductions achieved by these rehabilitation projects are not yet complete.

Milwaukee Metropolitan Sewerage District

The Milwaukee Metropolitan Sewerage District serves 28 communities in the southeastern portion of Wisconsin, the largest of which is the City of Milwaukee. The total service area includes over 2,800 miles of sewer mains, of which approximately 20 percent are of the combined storm/sanitary type, mostly located within the City of Milwaukee. The remaining 80 percent of the District is served by separate sanitary sewer systems, which were studied under a comprehensive SSES. Problems caused by high rain induced extraneous flows have included overflows and bypasses from the interceptor and collection systems, wastewater back-ups into building basements, and discharges of inadequately treated wastewater to Lake Michigan.

The field investigations conducted in the separate portion of the sewer system documented the following:

- o High peak flows occur in the system, of which 76 percent could be contributed to rain induced I/I (RII + SWI). The ratio of PWWF to ADWF was projected to be about 7.5 to 1.
- o Numerous defects were found in manholes from smoke testing and physical inspection programs.
- o Manhole frame/chimney defects were found to contribute significant flows based on street flooding studies to simulate rainfall conditions.
- o Numerous direct foundation drain connections were identified through building inspections.
- o Indirect connections between storm drains and sanitary sewers were found by smoke testing and dye flooding programs.
- o Foundation drains and building laterals exhibited peak flow responses to rainfall and to experimental rainfall simulation.
- o Approximately 60 percent of peak extraneous flow was attributed to RII, including 40 percent through foundation drains and 12 percent through manhole frame/chimney joints.

The high rain induced flow response and the presence of sewer system defects and foundation drain connections that accounted for 60 percent of peak extraneous flows indicate that RII is a significant problem in the system. The key factors affecting RII appear to be the prevalence of foundation drain connections, storm and sanitary sewer laterals constructed in the same trench in many areas of the system, high groundwater, and frost heave. Frost heave, or lifting and distortion of the ground surface due to subsurface ice formation, is believed to be a major factor in the formation of manhole frame/chimney defects and the cracks in concrete pavements that generate pathways to these defects.

Problem Assessment

As a result of its SSES, the District has conducted I/I correction work, primarily aimed at eliminating direct inflow through manhole covers and RII from manhole frame/chimney interfaces. The District conducted a manhole rehabilitation pilot project to evaluate different methods of correcting manhole frame/chimney leakage. Two of the District communities have successfully implemented foundation drain disconnection programs. A permanent monitoring system is being installed for long-term monitoring of I/I flows throughout the District.

Northeast Ohio Regional Sewer District

The Northeast Ohio Regional Sewer District includes 41 communities in the Cleveland, Ohio, metropolitan area. The District is divided into two major subdistricts: The City of Cleveland, which has a combined sewer system; and the surrounding communities, which primarily have separate systems. Most of the separated portions of the system are contained within two major planning areas, the Easterly Separate Sewer Area and the Southwest Interceptor Area which together contain approximately 1,200 miles of sanitary sewers serving a population of about 500,000.

Overflows and bypasses occur at over 200 locations in the separated sewer systems, most initiated by rain events of less than 0.2 inches per hour. Pump stations and regulator chambers in the interceptor system are used to restrict flow to the WWTPs. Basement back-ups are a major problem during wet weather.

Field investigations in the separate sewer areas documented the following:

- High flows occurred in the system in response to rainfall. The ratios of PWWF to ADWF was projected to be about 12 to 1 in the Southwest Area and over 20 to 1 in the Easterly Area.
- Identified direct inflow accounted for only 5 to 15 percent of the peak extraneous flow.
- Sanitary and storm sewers and building laterals were constructed in common trenches in over 50 percent of the separate system.
- Indirect flow transfer from storm to sanitary sewers was found to be very rapid, as documented by dye flooding tests.

These findings indicated that RII is a significant problem in the sewer system. The most significant factors affecting RII appear to be the poor condition of the sewers and laterals and the extensive common trench storm/sanitary system. Therefore, storm drain exfiltration appears to be the primary pathway for RII into this system.

RII correction efforts in the District have primarily concentrated on rehabilitation and flow regulation in the storm sewer system, with some sanitary sewer rehabilitation. Work has included construction of new storm sewers to replace common trench facilities and provide additional storm drainage capacity; rehabilitation of common trench storm/sanitary sewer manholes; and installation of vortex regulators to restrict flow into the storm drain system and thereby reduce the transfer of flow to the sanitary system.

City of Baton Rouge, Louisiana

The City of Baton Rouge is located in the southeast portion of Louisiana along the Mississippi River. Its sewer system serves a population of about 450,000 and includes approximately 1,500 miles of mains. The system is divided into four major areas, three of which comprise the original Consolidated Sewer District and the fourth, the suburban area. Each of the three original Consolidated Sewer District areas is served by its own WWTP; the suburban area includes 144 local wastewater treatment facilities. Overflows and bypasses have occurred throughout the sewer system during high intensity storm events.

Findings of field investigations were as follows:

- High peak flows occurred in the system in response to rainfall. The overall ratio of PWWF to ADWF is estimated to be about 3.5 to 1.

- Numerous defects were detected in sewer mains, manholes, and building laterals through smoke testing programs.

- Most potential transfers of water from storm drains to sanitary sewers were found to be through defects in the sewers.

- In four special study areas, building lateral defects were found to account for 32 percent of the potential peak rain induced extraneous flow, with the remainder coming from sewer mains and manholes.

The high rain induced flow response, the absence of direct inflow connections to account for any substantial portion of the peak flows, and the prevalence of defects in sewer mains and laterals indicated that RII is a significant component of peak wet weather flows in the Baton Rouge sanitary sewer system. The key factors affecting RII appear to be the poor condition of sewers and laterals, due to both age and lack of system maintenance; construction of sewer trenches in drainage ditches; and the shallow depth of building laterals.

The City is implementing a rehabilitation program to correct all main line defects identified during field testing in the four special study areas. Rehabilitation techniques will include spot repair, pipe replacement, slip-lining, and manhole sealing.

City of Springfield, Missouri

Springfield is located in southwestern Missouri. The wastewater service area is divided into two main drainage basins, each served by a separate WWTP. The larger of the two basins is the Southwest area, which includes approximately 80 percent of the City. This area includes over 500 miles of sanitary sewers, which serve approximately 160,000 people.

Identified problems due to rainfall induced extraneous flows include surcharging of and overflows from the collection system, as well as basement flooding. Overflows occur at approximately ten sites during any good-sized storm, and at 100 or more locations during large rainfall events. I/I correction efforts aimed at eliminating direct inflow and the repairing of isolated problem sewer reaches did not have a noticeable impact on peak wet weather flows.

Findings of limited field investigations included:

- High flows occurred in response to rainfall, with the ratio of PWWF to ADWF estimated to be about 8 to 1. Larger, longer duration storms produced higher and more sustained peak flows than short-duration, thunderstorm-type events.
- Relatively few direct inflow connections were found through smoke testing programs.
- Evidence of infiltration through manhole walls and inverts was observed during physical inspections.
- Clear water discharges from laterals were observed during TV inspection work.
- Many sewers were installed in the shallow limestone bedrock, which supports a perched groundwater table in much of the area.

The high rain induced flow response and the failure of the inflow correction program to reduce rain induced extraneous flows indicated that RII is probably a significant component of peak wet weather flows in the system. The key factors affecting RII appear to be the age and poor condition of the sewers and the hydrogeologic conditions characteristic of the service area, which are conducive to rapid transport of water to sewer defects.

The City has conducted sewer grouting in the system since 1972, primarily concentrated in older areas. A pilot project in a newer area was also conducted, with sewer main grouting and manhole sealing. No significant flow reductions were achieved through these efforts. The City has implemented a long-term correction program involving routine TV inspection and rehabilitation of sewer mains on a priority basis, primarily by slip-lining.

North and South Shenango Joint Municipal Authority, Pennsylvania

The North and South Shenango Joint Municipal Authority includes the Townships of North and South Shenango, which are located along the shoreline of Pymatuning Reservoir in northwestern Pennsylvania. The Authority operates a collection system and treatment plant, which serve a permanent population of about 1,200 and a summer population of approximately 6,000. The collection system includes approximately 90 miles of sewer mains and several pump stations.

The sanitary sewer system was originally constructed in 1978. Although the contract specifications for the sewer system included strict criteria for maximum allowable infiltration, wet weather flows in the system have far exceeded design capacity, resulting in overflows at the pump stations and hydraulic overloads of the WWTP. Major wet weather problems have occurred in four areas of the system that were installed under one construction contract and with clay pipe made by a different manufacturer than that installed in other portions of the system.

Findings of field investigations were:

- High flows occurred in response to rainfall, with sustained peak flows after the end of rainfall. The estimated ratio of PWWF to ADWF is about 7 to 1.

- High groundwater exists in much of the area, and a large portion of the sewer system is submerged.

- Rapid increases in water levels in sewer trenches from rainfall occurrences were noted through monitoring water levels in the trenches.

- The rate of infiltration into individual pipe joints was found to increase directly with the depth of water over the pipe.

- o Many sewers were constructed directly under area drainage ditches.
- o Limited smoke testing detected no significant direct inflow connections.
- o Very little extraneous flows from building laterals was observed during TV inspection of sewer mains.

Sewer flow and trench water level responses to rainfall, as well as the absence of direct inflow connections confirmed that the high rain induced extraneous flows in the North and South Shenango sewer system are due to RII. Entry points of RII appear to be primarily through defective pipe joints. The other key factors which affect RII are the construction of sewer trenches in drainage ditches and the high groundwater level in the service area.

To correct the RII problem, the Authority is slip-lining all of the sewer mains and slip-lining or replacing the lower laterals in the four problem contract areas (the upper laterals are constructed of PVC pipe and are not believed to contribute RII). Limited rehabilitation work conducted prior to the full-scale rehabilitation effort indicated that grouting would not be effective in eliminating infiltration through the pipe joints. A pilot slip-lining project, however, appeared to achieve virtually complete elimination of extraneous flows.

City of Ames, Iowa

Ames is located in central Iowa along the Skunk River. The collection system, containing approximately 135 miles of sewer mains, serves a population of approximately 45,000, almost half of which comprise the Iowa State University campus.

During wet weather periods, the WWTP cannot treat all of the peak flows in the system. An influent sluice gate must be throttled, often for as long as several days, to limit flow entering the plant. Several times each year during extremely wet conditions, bypassing of raw wastewater occurs both at the plant and at several points in the collection system. Basement backups during wet weather also occur as a result of high wet weather flows.

Findings of field investigations were as follows:

- o High peak flows occurred in response to rainfall. The ratio of PWWF to ADWF is estimated to be about 6 to 1.
- o Identified direct inflow accounted for about 40 percent of the peak extraneous flow.

- o A survey identified 1,800 direct foundation drain connections to the sanitary sewer system. Many additional potential connections, where foundation drains incorporated valving to divert flow to the sanitary sewer during freezing conditions, were also identified.

- o Foundation drains were found to exhibit peak flow responses based on a study of the impact of simulated rainfall on foundation drain sump pump operating times.

- o The flow from foundation drains was estimated to account for about 50 percent of peak extraneous flows in the system.

The high rain induced flow response, the existence of many directly connected foundation drains, and documentation of the peak flow response from foundation drain discharges to rainfall, indicated that RII is a significant component of peak wet weather flows in the Ames sanitary sewer system. High groundwater appears to be a key factor affecting the occurrence of RII through foundation drains.

As part of its overall I/I correction program, the City has implemented a foundation drain disconnection program targeted at eliminating 768 foundation drain connections over a ten-year period. The program involves a public information effort and includes provisions to reimburse a large portion of the homeowners' disconnections costs. Over 300 disconnections were achieved in the initial two years of the program on an entirely voluntary basis. The City anticipates that the program will continue beyond the required 768 disconnections.

City of Coos Bay, Oregon

Coos Bay is located on the southwest coast of Oregon. The wastewater system serves a population of about 15,000 and contains approximately 60 miles of sanitary sewers. The sewer system is primarily a separate system, although a small portion is believed to be partially combined. The City is divided into two main sewer service areas, each served by a separate WWTP. The major wet weather flow problems are concentrated in the collection system which serves the eastern portion of the City and an adjacent sanitary district. Problems due to high peak wet weather flows have included bypassing and overflows in the collection system, as well as bypasses of untreated wastewater and discharge requirement violations at the WWTP.

The occurrence of RII in the Coos Bay sanitary sewer system is indicated by the following:

- o From the early 1970's through 1982, field investigations and rehabilitation work to reduce extraneous flows were conducted, including disconnection of known direct inflow connections (downspouts and cross connections with the storm drain system) and a sewer main rehabilitation program.

- o Smoke testing conducted after the rehabilitation work confirmed that almost all direct inflow connections had been eliminated from the system.

- o High peak wet weather flows still occurred in the system after completion of the rehabilitation program. The ratio of PWWF to ADWF was projected to be about 8 to 1.

This evidence indicates that the peak rain induced flows in the sanitary sewer system appear to be due primarily to RII. The key factors affecting RII are the poor condition of the sewers, due in part to ground settlement in the bay mud which underlies much of the older portions of the system; the shallow depth of building laterals; and the high groundwater which characterizes the service area.

In previous years, the City has conducted rehabilitation (primarily grouting and some replacement) of sewer mains with major problems identified through TV inspection and smoke testing. As noted above, these efforts did not result in any significant reductions in wet weather flows. However, a program of routine TV inspection has been initiated to identify specific areas in need of repair or replacement.

City of Tulsa, Oklahoma

Tulsa is located in northeast Oklahoma along the Arkansas River. Total service area population is approximately 400,000, and the collection system includes over 1,400 miles of sewer mains. The City has conducted field investigations and rehabilitation of the sewer system since 1982, both as part of overall facilities planning efforts to reduce sanitary sewer surcharging and overflows during rainfall.

Field investigations documented that:

- o High peak flows occurred in response to rainfall. The ratio of PWWF to ADWF is estimated to be about 3.5 to 1.

- o Numerous defects were found in sewer mains, manholes, and service laterals from smoke testing programs.

- o Defects were observed in sewer mains through TV inspection efforts.

- o Direct inflow connections detected through smoke testing accounted for 30 percent of the estimated extraneous flows. RII accounted for the remaining 70 percent.

- o Of the potential flow contribution from RII entry points detected during smoke testing, about 45 percent was estimated to be from service laterals, 35 percent from sewer mains, and 20 percent from manholes.

- o The estimated flow contributions from direct inflow connections and sewer system defects identified through smoke testing could not account for all of the rain induced extraneous flows.

The high rain induced flows, the fact that direct inflow connections accounted for less than 30 percent of the peak extraneous flows, and the prevalence of defects in sewer mains, manholes, and service laterals indicated that RII is a significant component of peak wet weather flows in the Tulsa sanitary sewer system. The key factors affecting the occurrence of RII appear to be the poor condition of the sewers system, shallow depth of laterals, granular trench backfill, and the shallow limestone bedrock that characterizes the service area.

Rehabilitation was performed as part of the City's SSES. The rehabilitation work consisted primarily of slip-lining, inversion lining, pipe replacement, manhole sealing, and spot repairs of the public portions of the system (mains, manholes, and lower laterals), as well as disconnection of direct inflow sources. In general, only those specific defects detected through the SSES field work and determined to be cost-effective for correction were addressed. Voluntary repair of leaking private laterals and cleanouts was encouraged through a public relations program. For eight subbasins in which rehabilitation was performed, the initial reductions in peak wet weather flows were reported to average approximately 50 percent.

SUMMARY

- o RII is a form of infiltration into sanitary sewer systems characterized by a significant, short-term increase in flow in direct response to rainfall.

- o RII enters the sewer system from the ground through defective pipes and manholes and through foundation drains. RII entry points are similar to those of "classical" infiltration, or groundwater infiltration (GWI).

Problem Assessment

- The RII flow response may be indistinguishable from that of direct storm water inflow (SWI) if it is very rapid and short-termed.

- The RII flow response is likely a continuum from a very gradually changing flow, similar to GWI, to a rapid peak, similar to SWI.

- The traditional methodology for analysis of I/I has resulted in RII being incorrectly identified as inflow in many sewer systems.

- Peak wet weather flows due to RII can cause overflows and bypasses in sanitary sewer systems and at wastewater treatment plants, as well as backups of wastewater into building basements. Peak wet weather flows include base wastewater flow plus GWI plus rain induced infiltration and inflow.

- To handle RII flows, sewer pipelines and pump stations and wastewater treatment plants must be designed with considerable additional capacity to convey and treat relatively infrequent, but large peak flows.

- Estimated RII ranged from over 50 to nearly 100 percent of total peak rain induced extraneous flow for the ten case studies documented in this investigation. Rain induced extraneous flow includes only rainfall infiltration and inflow.

- Possible pathways of storm water flow from the ground surface to the sanitary sewers may include:

  - Soil channels from the ground surface to sewer defects.

  - Exfiltration out of leaky storm drains through the soil to defects in sanitary sewer pipes.

  - Seepage through pavement cracks with horizontal movement along the street subbase to the upper portions of sanitary sewer system manholes.

  - Percolation into permeable trench backfill materials and along pipe trenches to defects in sewer pipes.

- RII was found to enter sanitary sewers through pipe defects in sewer mains and building laterals, manhole defects, and foundation drains directly connected to service laterals.

Problem Assessment

- Several factors were found to be significant in the formation of RII entry points.

  - Age of the sewer system.
  - Type of pipe and joint materials.
  - Construction practices.
  - Lack of proper maintenance.
  - Freeze/thaw conditions.
  - Earth movement.
  - Root intrusion.
  - Lack of ordinance enforcement prohibiting foundation drain connections.

- Defective building laterals and foundation drain connections on private property can contribute significant RII flows, and even the majority of RII in some systems.

- Different types of RII entry points appear to predominate in various geographical areas of the U.S.

  In areas where foundation drains are common, such as the midwest, foundation drain connections can contribute a major portion of the RII. Where sewer mains and laterals are relatively shallow, such as in the western and southern portions of the U.S., pipe defects may be the predominant RII entry points.

- For the systems reviewed in this study, the projected overall system peak wet weather flow (PWWF) to average dry weather flow (ADWF) values ranged from about 3.5 to over 20.

  However, projected peak flows are not necessarily directly comparable because they are based on design storm criteria specific to each system and also because they typically include at least some component of SWI and GWI.

- It is likely that RII can occur to some extent in any sewer system and can be a significant component of wet weather flows.

  While the sample of RII case studies evaluated was not large enough to conclusively determine the national significance of the RII problem, sewer system and environmental factors which appear to influence the occurrence of RII can be found in systems throughout the country.

Problem Assessment

- Problems associated with defective sewer pipes are not limited to RII. Exfiltration of untreated sewage through these defective pipes may contaminate ground-water. This problem is likely to manifest itself when the sewer pipe is above the water table.

TABLE 2-1

SUMMARY OF CASE STUDIES

System Name	System Size[a] (miles)	Points of RII Entry	Probable Factors Affecting: Points of Entry	Probable Factors Affecting: Flow Response	PWWF/ADWF[b] Ratio	Control Efforts
EBMUD, CA	1,500	• Sewer main defects • Service lateral defects	• Age, condition of sewers. • High density • Poor maintenance • Ground movement	• Shallow mains and laterals • Clay soils • Steep slopes	20	"Comprehensive" rehabilitation (mains plus entire laterals) in about 50 percent of service area; primarily replacement and slip-lining.
Springfield, OR	165	• Sewer main defects • Service lateral defects	• Condition of sewers	• High groundwater	11	"Complete basin" rehabilitation (mains plus lower laterals) in four basins; primarily replacement and grouting.
MMSD, WI	2,200	• Foundation drain connections • Manhole frame/ chimney defects • Sewer main defects • Service lateral defects	• Foundation drain connections • Frost heave	• Common trench laterals • High groundwater	7.5	Manhole rehabilitation. Foundation drain disconnection in two communities only.
NEORSD, OH	1,200	• Sewer main defects • Service lateral defects • Common trench manhole dividers (walls, plates)	• Condition of sewers	• Common trench storm/ sanitary sewer construction	12-20+	Common trench sewer separation and manhole rehabilitation. Vortex regulators to restrict storm drain flow.
Baton Rouge, LA	1,500	• Sewer main defects • Service lateral defects	• Age, condition of sewers • Poor maintenance	• Trenches in drainage ditches • Shallow laterals	3.5	Rehabilitation in four pilot areas, primarily slip-lining, grouting, and replacement of sewer main defects identified through smoke testing.

TABLE 2-1

SUMMARY OF CASE STUDIES (CONTINUED)

System Name	System Size[a] (miles)	Points of RII Entry	Probable Factors Affecting: Points of Entry	Probable Factors Affecting: Flow Response	PWWF/ADWF[b] Ratio	Control Efforts
Springfield, MO	500	• Sewer main defects • Service lateral defects • Foundation drain connections	• Age, condition of sewers	• Shallow bedrock (limestone) • Perched groundwater	8	Long-term, routine inspection and rehabilitation of mains on priority basis.
N&S Shenango, PA	90	• Pipe joints	• Defective pipe joints	• Trenches in drainage ditches • High groundwater	7	Slip-lining of sewer mains and lower laterals.
Ames, IA	135	• Foundation drain connections	• Foundation drain connections	• High groundwater	6	Foundation drain disconnection program.
Coos Bay, OR	80	• Sewer main defects • Service lateral defects	• Condition of sewers • Ground settlement	• Shallow laterals • High groundwater	8	Limited sewer main rehabilitation.
Tulsa, OK	1,400	• Sewer main defects • Service lateral defects • Manhole defects	• Condition of sewers	• Shallow laterals • Shallow bedrock (limestone) • Granular trench backfill	3.5	Rehabilitation in selected subbasins; primarily lining, replacement, and spot repairs of defects identified through field work.

a Separated portion only, if partially combined system.
b PWWF = Peak Wet Weather Flow; ADWF = Average Dry Weather FlowPWWF may include varying amounts of SWI and GWI, and is typically based on a specified design storm for each system. Therefore, PWWF/ADWF ratios are not necessarily comparable between systems.

CHAPTER 3

CONTROL METHODS

This chapter discusses different methods and approaches for controlling rainfall induced infiltration into sanitary sewer systems. The basis of the discussions was a literature search to identify sewer system rehabilitation methods currently being practiced both in the U.S. and in other parts of the world. In addition, the discussions draw upon information collected as part of the RII candidate system search and case study documentation presented in Chapter 2. A description of an "example" RII control program, that of EBMUD, is also presented.

"Control" means the implementation of methods to reduce existing RII flows or limit future RII into a sewer system. RII control can be accomplished through physical rehabilitation of existing sewers and application of proper design standards and construction practices for new sewers. Institutional and regulatory approaches and preventive maintenance programs are means of facilitating implementation and ensuring the effectiveness of RII control programs. The success of an RII control program is dependent not only on the application of appropriate engineering techniques, but also on the overall implementation approach used.

Typically, control methods are aimed at correcting the entry points of extraneous flows into the sewer system. Physical methods to rehabilitate sewers are largely applicable to all types of infiltration (GWI and RII). However, RII control differs from GWI control in the approaches used to quantify flows and identify entry points, and the relative importance that is placed on correction of particular components of the sewer system (e.g., mains versus laterals). For this reason, this chapter also includes a discussion on field investigation techniques to quantify and identify RII, as well as various approaches for implementing RII control programs.

RII FIELD INVESTIGATION TECHNIQUES

In general, the same traditional methods that have been used for conventional I/I investigations and SSESs can be used as field techniques for RII investigation. However, to identify and quantify RII, it is critical that the field methods be appropriately applied and the data be properly interpreted. The field techniques and methods of data interpretation, as they specifically apply to RII, are discussed below.

Flow Monitoring

Flow monitoring is commonly used to quantify I/I flows in different portions of a sewer system. To obtain accurate and useful flow data, no hydraulic constrictions should exist in the vicinity of the flow monitor, and the sewers upstream and downstream of the monitoring site should be cleaned prior to monitoring to remove major root intrusion and sediment buildup. The monitoring manhole ideally should have smooth transitions and no side streams or changes in flow direction. Collection of useable flow data can be better assured by appropriate choice of monitoring equipment (e.g., depth-velocity meters versus level-only recorders), as well as suitable monitor location. Surcharging or backwater effects must be carefully evaluated in interpreting flow monitoring results.

The traditional approach for analyzing wet weather flows is to subtract the non-rainfall base flow (base sanitary flow plus GWI) during the period immediately preceding a storm event from the total flow during and immediately following the storm. The difference is the rainfall induced infiltration and inflow (RII+SWI). An adjustment can be made to account for the higher sustained GWI rate at the end of the storm (see Figure 2-1). However, it is generally impossible to distinguish SWI from RII on the basis of this hydrography alone.

One approach to interpreting the (RII+SWI) hydrography is to separate it into component parts, each representing a different response time to rainfall. This type of analysis can be used to identify the relative significance of different types of RII by the relative magnitude of each hydrography component. The most rapid component (shortest time to peak) will include the SWI portion of the flow, as well as some portion of the RII. The slower components will typically consist of RII only.

If the flow monitoring period is long enough to include different types of storms (e.g., short, intense storms and extended duration storms), then a comparison of (RII+SWI) hydrographs may also indicate the relative significance of RII in the system. In some systems, it has been found that longer-duration storms and/or those characterized by considerable antecedent rainfall produce higher peaks and larger RII volumes than comparable isolated, short-duration storms.

Control Methods

Flow Isolation

Flow isolation or flow mapping is a technique commonly used to determine the relative I/I contribution from different "minibasins" or reaches of sewer within a subsystem. The procedure consists of taking instantaneous manual flow measurements at successive manholes. For RII isolation, flow measurements are taken during and immediately after rainfall.

Since the objective of RII flow isolation is to determine the peak RII contribution at different locations in the sewer system, care must be taken in comparing measurements from different locations taken at different times during the rainfall. One way to help overcome this difficulty is to place a continuously recording flow monitor at a location downstream in the subsystem. The instantaneous measurements taken within the subsystem at various times during and after the rainfall are then projected to a peak flow (assumed to occur at the same time as the peak of the downstream monitor hydrograph). The projection is based on the ratio of the monitor flow at the measurement time to the monitor peak flow. If flow isolation is conducted several hours after the peak rain period but still while the flows are elevated above normal levels, the measured flow can reasonably be assumed to consist primarily of RII because the SWI flow hydrograph should already have receded.

Flow isolation during rainfall is an effective way to determine the distribution of RII in the subsystem. This allows rehabilitation efforts to be concentrated in those minibasins with relatively higher RII contributions. Equally important, any losses in flow between successive manholes indicate exfiltration and therefore, appropriate corrective measures for the problem should be considered.

Groundwater Monitoring

Groundwater monitoring is used to determine the elevation of the groundwater with respect to that of the sewer system and its long-term or short-term variations. Groundwater monitors placed within a sewer trench provide a direct measurement of the hydrostatic pressure on defects in the sewer pipe. Continuously recording monitors can be used to determine short-term responses to rainfall, which can then be correlated with flow measurements in the sewers. Ground-water monitoring can also be used to monitor the quality of ground-water when exfiltration is determined to be a serious problem.

Smoke Testing

Smoke testing is the traditional field method used to detect direct inflow entry points. Under appropriate conditions, it can also identify some types of RII entry points. Specifically, for smoke testing to be an effective RII investigation technique, the sewer must be above the groundwater level and the soil must be relatively dry. Under these conditions, smoke will be transmitted through channels in the soil and be detected as visible emissions from the ground over defective sewer pipes, laterals, and manholes, or from storm drains or catch basins (in the case of RII due to exfiltration from storm drains).

In general, detection of RII entry points by smoke testing will be most successful in cases where most of the defects are close to the ground surface (i.e., shallow mains and laterals) and where there are relatively few direct inflow connections (since these sources would tend to draw most of the smoke). The absence of smoke from potential RII entry points does not mean that they do not exist; some pipes may have traps or sags that prevent smoke travel. However, the defects that do emit smoke are likely to be those with the most rapid flow response to rainfall.

Dye Flooding

Dye flooding is generally used to verify direct and indirect connections between storm drains and sanitary sewers. A storm drain or ditch is flooded with dyed water, and the sanitary sewer is observed for appearance of the dye in the flow stream. The flow rate and concentration of the dye gives an indication of whether the connection is direct or indirect. TV inspection of the sanitary sewer concurrent with dye flooding provides direct evidence of specific locations where RII enters the sewer. If the storm sewer is completely flooded (surcharged) during the dye flooding, the rate of flow into the sanitary sewer (or into individual defects) may approximate the peak RII flow during a large storm.

Street Flooding

Street flooding can be used to identify and quantify RII flows into such entry points as manhole frame/chimney defects. Surface water is prevented from entering the manhole by placement of an inner tube in the frame opening, which still permits visual observation of the flow entering from the ground below the frame. The leakage rate through the manhole frame/chimney defect is measured or estimated based on observation. Leakage rate under this "simulated" rainfall condition is assumed to approximate the RII flow.

Rainfall Simulation

Rainfall simulation consists of applying water to an area of suspected RII and observing or measuring the resulting flow.

Rainfall simulation is typically used on a limited basis to estimate flows from foundation drain connections or defective building laterals. The results of rainfall simulation provide evidence as to the potential magnitude and speed of the flow response to rainfall from these system components. Note that RII flows during actual rainfall events can also be observed or measured in a similar manner.

Manhole and TV Inspection

Physical inspection of manholes and internal television inspection of sewers are used to identify defects in sewer pipes and manholes which can be potential entry points for extraneous flows. Material deposits and stains, often indicators of infiltration, can also be observed. If conducted during rainfall, manhole and TV inspection can identify specific entry points of RII. TV inspection as a RII detection technique is limited because many sewers become surcharged during rainfall conditions, thereby preventing observation of RII entry to the system. Also, an apparently good sewer (no observed defects) does not necessarily mean that RII entry points do not exist. Quite often, infiltration occurs below the wastewater flow line or at joints in the sewer pipe that cannot be seen by the camera.

TV inspection is relatively expensive and generally should be used only after a specific sewer reach has been identified through flow isolation or dye flooding as contributing significant RII flows. Its use for inspection of building laterals provides the same type of information, but lateral TV inspection generally requires special "mini-cameras" which can crawl or be pushed up the lateral. Lateral TV inspection is also limited by the availability of access points (cleanouts).

Building Inspection

Physical inspection of building basements is used to identify direct foundation drain connections to the sanitary sewer system. Floor drains are inspected for evidence of a connection with the foundation drain (via a Palmer valve or drain tile receiver). Sump pump discharge points are also determined during building inspections.

SEWER REHABILITATION METHODS

Rehabilitation refers to physical repairs or modifications to sanitary sewer system components which can reduce the amount of RII entering the system. Sewer rehabilitation as a RII control method is generally aimed at eliminating RII entry points, specifically, pipe and manhole defects and foundation drain connections. The selection of an appropriate rehabilitation technique to repair any specific sewer pipe, lateral, or manhole is a design decision that must be based on existing structural condition, type of defects, site constraints, and cost considerations. All of the rehabilitation methods described in this section are applicable for RII control; the "best" method for any particular situation will depend upon the factors listed above.

The effectiveness of sewer rehabilitation in reducing RII depends not only on the proper selection and application of rehabilitation technique, but also, and primarily, on the overall rehabilitation program approach. As discussed later in this chapter, rehabilitation programs which address only isolated, large defects or only the public portion of the sewer system may be ineffective in reducing RII. Area-wide rehabilitation, including private as well as public facilities, is generally necessary to achieve significant RII reductions.

Sewer rehabilitation methods range from complete replacement or construction of new facilities to repairs of individual defects that can be accomplished in place. In general, the costs for complete replacement are significant, especially when based on not only the cost of construction but also the indirect costs resulting from construction. These indirect costs have been a driving force for the development of less expensive, less physically disruptive techniques for in-place rehabilitation. The following paragraphs briefly review the various techniques available for sewer system rehabilitation to reduce RII. More detailed descriptions and discussions of these methods are included in Appendix D.

Pipeline Rehabilitation

Rehabilitation methods for sewer pipelines include conventional and trenchless replacement, grouting, and several different lining techniques. The rehabilitation techniques listed below are not all-inclusive; other techniques are currently being developed.

The focus in pipeline rehabilitation today is on in-place techniques such as lining and trenchless replacement. These methods minimize the impact on traffic, other utilities, and surface improvements. One of the main shortcomings of the in-place techniques is making a leak-free connection between the main and lateral without excavating. Because these connections are often responsible for significant leakage, the effectiveness of the seal at this joint may be essential to RII reduction.

Many of the techniques originally developed for sewer mains have been modified for lateral rehabilitation. However, since laterals are typically short (less than 75 feet), may have many bends or offsets, and often lack useable points of access, their rehabilitation by in-place techniques is generally less cost effective than for mains. Access to laterals, both for testing and rehabilitation, is also an institutional problem, primarily because the installation and maintenance of laterals are usually the legal responsibilities of the property owner.

The following techniques are applicable for rehabilitation of sewer pipelines:

Conventional Replacement. Conventional replacement can be used as a method for rehabilitation of a complete manhole-to-manhole pipe reach, as well as for repair of individual defects. The replacement of an entire reach using modern pipe materials provides an essentially leak-free pipe. Excavation and repair of isolated, joint-to-joint pipe sections (point repairs) may often be required in conjunction with other sewer rehabilitation techniques such as grouting or lining. Lateral to main connections also generally require excavation for restoration of a leak-free joint.

Trenchless Replacement. Tunneling and moling are methods of trenchless installation of new pipe. Variations of some of these techniques, such as that commonly referred to as "pipe bursting," can be used to replace a pipe along its existing alignment, including installation of a larger diameter pipe. Flexible, jointless pipe, such as polyethylene, is an effective replacement material for RII control. Moling is often attractive for laterals to minimize surface impacts and allow the existing lateral to remain in service until the new service is installed. Also, new construction using these techniques does not require granular backfill, thereby minimizing the potential for transfer of extraneous water into and along the sewer trench.

Grouting. Grouting is used to seal joints, small holes, and radial cracks in otherwise sound pipe. Pipes in poor structural condition or with numerous defective lateral connections generally cannot be effectively repaired by grouting. Grouting requires no excavation where manhole entry is available. The long-term effectiveness of grouting depends upon the type of grout used, the moisture conditions around the pipe, and proper application and quality control. Periodic testing after the initial grouting may be required, not only to re-test the seal on the grouted joints, but also to correct new leaks in previously ungrouted joints and cracks.

Slip-lining. Slip-lining consists of inserting a new liner pipe inside an existing sewer pipe or lateral. The liner pipe, typically high-density polyethylene, can be fused into long, joint-free (and therefore, leak-free) sections prior to insertion. Grouting must be used to seal the annular space between the liner and existing pipe at manholes, and may be used to seal the annular space for the entire length of the pipe reach. Some newer slip-lining methods utilize short, threaded liner pieces, helically wound strips, and expandable liners to facilitate the insertion process. When a sewer main is slip-lined, each lateral connection must be excavated and reconnected to the slip-lined pipe. If the laterals are also slip-lined, the lateral and main sewer liners can be fused together to make a leak-free joint.

Cured-in-Place Lining. Cured-in-place lining utilizes a thermal-setting, resin-coated flexible fabric liner. The liner is typically inserted in the pipe by inversion. Once inserted inside the pipe, the liner is hardened by circulation of hot water or steam. The liner conforms to the internal shape of the existing outer pipe and provides a smooth, joint-free lining. Although a remote cutting device is available to reconnect laterals to the lined pipe, remote cutting does not provide any means of sealing these joints. Therefore, if the lateral connections are subject to leakage, they must be excavated for repair as in slip-lining. Cured-in-place lining requires less surface excavation than does conventional slip-lining, but is generally more expensive.

Manhole Rehabilitation

Specific manhole rehabilitation techniques are designed to correct manhole frame/chimney defects as well as to eliminate RII entering through the walls and base. The Milwaukee Metropolitan Sewerage District has pioneered the development and testing of several new repair techniques as part of its manhole rehabilitation pilot program. Rehabilitation methods for manholes include both interior and exterior techniques. Interior repairs are generally less expensive and time consuming, but are frequently less effective.

Interior Repair Methods. Interior repair methods, although typically less effective for RII control, remain attractive in many cases due to the low cost and ease of undertaking. These techniques make possible the sealing of all manhole joints, including the lower ones, which are often subject to the largest hydrostatic forces. Interior repair techniques utilize:

- o Elastomeric sealants.
- o Chemical grouts.
- o Internal boots.

Exterior Repair Methods. Exterior repairs are often more effective than internal repair methods, but require excavation. Since it is difficult to gain access to all manhole joints, external repairs generally focus on the joints close to the ground surface, including the manhole frame/chimney connection. Exterior repair methods
utilize:

- Elastomeric sealants.
- Elastomeric sheeting.
- Rubber sleeves.
- Two-piece frames.

Foundation Drain Disconnection

Methods for foundation drain disconnection are relatively straightforward and depend primarily on the configuration of the existing connection. The disconnection involves:

- Directing the foundation drainage discharge to a sump.

- Installation of a sump pump.

- Construction of a discharge line to the outside of the building or to a storm drain.

- Plugging the existing connection to the sanitary lateral.

If the sump and/or sump pump already exists, then the disconnection may simply involve redirecting the discharge to an appropriate point. If the discharge is to go to a storm sewer, connection to an existing storm lateral or construction of a separate storm lateral to connect into the storm sewer may also be required.

DESIGN STANDARDS AND CONSTRUCTION PRACTICES

Effective design standards and construction practices can ensure minimization of the potential for RII in new sewer mains, manholes, and building laterals. Such standards and practices are also important for existing sewer system rehabilitation. This section presents the key concepts for design and construction as they apply to RII control. More detailed discussion of these issues are presented in Appendix E.

Modifications of sewer design standards provide a means of controlling future RII in sewer systems by preventing potential development of defects and minimizing the potential for the migration of extraneous water to any sewer defects which may develop. Such modifications include:

- Restricting the flow of water in granular backfill.

- Reduction of utility trench backfill interconnections.

- Control of migration of fine soil or backfill material particles.

- Reduction in the number of pipe joints.

- o Incorporation of pipe system flexibility to reduce settlement stresses.
- o Improved sealing of pipe connections at manholes.
- o Provision for tight, but flexible, lateral connections.
- o Provisions for access for testing, inspection, and repair of laterals.

Implementation of effective sewer construction practices ensures that design standards are properly addressed. This is accomplished by regular construction inspection and adequate performance testing, both for public sewer mains and manholes and private building laterals. Leakage tests (air pressure or water) must include stringent standards that assure an acceptable infiltration rate over the life of the sewer. Tests that allow for relatively large leaks from individual joints, even though the overall leakage in the pipe reach does not appear to be excessive, may not be acceptable.

RII CONTROL PROGRAM APPROACHES

Various approaches have been taken to control infiltration and inflow into sanitary sewer systems, but few have specifically addressed RII alone. Typical control programs have consisted of physical rehabilitation of portions of the existing sewer system in an attempt to immediately reduce the magnitude of extraneous flows. Rehabilitation projects may have included some private facilities (building laterals or foundation drains), but typically have only addressed the public portion of the system. Long-term control programs have sometimes been initiated either in lieu of or in conjunction with immediate large-scale rehabilitation efforts.

The most controversial aspect of control programs is the question of how to deal with problems on private property. In recent years, many communities have realized that private property sources often contribute the majority of extraneous flows to the sewer system. Therefore, significant flow reductions can only be achieved if sources on private property are also addressed by the control program. However, rehabilitation on private property entails institutional, financial, and construction problems that are often perceived to be prohibitive.

This section discusses various <u>approaches</u> to RII control. In this context, approaches imply various options for developing an overall control program, as opposed to selecting specific rehabilitation techniques or design standards. The latter two involve primarily engineering judgements. Selection of an appropriate and effective overall approach to RII control involves both engineering and institutional decisions. Institutional and regulatory approaches, discussed later in this chapter, are means of facilitating implementation of RII control programs.

Rehabilitation Program Approaches

In developing a rehabilitation program that will effectively achieve reductions in RII peak flows, it is essential to correctly identify the areas of the system and the types of entry points that must be corrected. The first step in any rehabilitation program should be to eliminate obvious sources of direct inflow. The reasons for this approach are:

- Direct inflow sources can be detected easily by smoke testing and are generally cost effective to remove.

- Once inflow sources are removed, RII can be quantified from flow monitoring (otherwise it is not possible to separate the SWI and RII portions of the rain induced I/I hydrograph), and those areas of the system which contribute significant RII flows can be readily identified.

Once the areas to be addressed in the RII control program are established, a proper approach for identifying the particular sewer system components to be rehabilitated must be developed. Approaches to rehabilitation programs may differ in the following basic ways:

- Addressing entire areas of the sewer system versus repair of individual defects only.

- Including both the private and public portions of the sewer system versus only the public portion.

Rehabilitation programs that have only addressed individual large defects or only problems on public property have often failed to achieve projected RII reductions. One of the reasons is that migration of RII to unrepaired defects can occur when only some RII entry points are eliminated. Furthermore, building lateral defects and/or foundation drain connections on private property may represent a significant portion of the RII in many systems. Therefore, the effectiveness of a rehabilitation program in reducing RII is dependent not only on the repair techniques used but also on the extent of the rehabilitation effort.

Long-term RII Control Approaches

Although a rehabilitation program may be effective in immediately reducing RII levels in a sewer system, it will not necessarily guarantee that those RII levels are maintained. Long-term control requires that RII be prevented from increasing in unrehabilitated areas of the system, as well as from entering from newly constructed sewers. Long-term RII control can be implemented through:

- Effective preventive maintenance programs.
- Implementation of appropriate design standards and construction practices.

An effective preventive maintenance program should include:

- Periodic flow monitoring in the system to identify areas with increases in RII levels.
- A routine program of cleaning and root removal.
- A cyclic program of testing and inspection of the sewers throughout the system to identify the need for repairs replacement.

In systems where defective building laterals or foundation drains represent a significant portion of the RII, the program should also include private facilities.

COST EVALUATION

Costs for sewer system rehabilitation to reduce RII must be compared to those for transport and treatment of RII flows to evaluate the cost effectiveness of various RII reduction options. The "traditional" approach to performing I/I cost-effectiveness analyses, as described in the EPA Handbook for Sewer System Evaluation and Rehabilitation, is based upon determining the flow contribution and correction cost for each individual I/I source in the sewer system identified through field inspection and testing. The I/I sources are then ranked in order of least cost per unit of I/I flow removed. The cumulative flow reduction and corresponding correction cost for successive elimination of the individual I/I sources in order of least unit cost ranking are calculated. The cumulative correction cost is then plotted against cumulative I/I removed, along with the corresponding (decreasing) cost for transport and treatment (see Figure 3-1). The low point of the total cost curve represents the cost-effective level of I/I reduction for the system. Those individual sources which rank above this level are considered to be cost effective to correct.

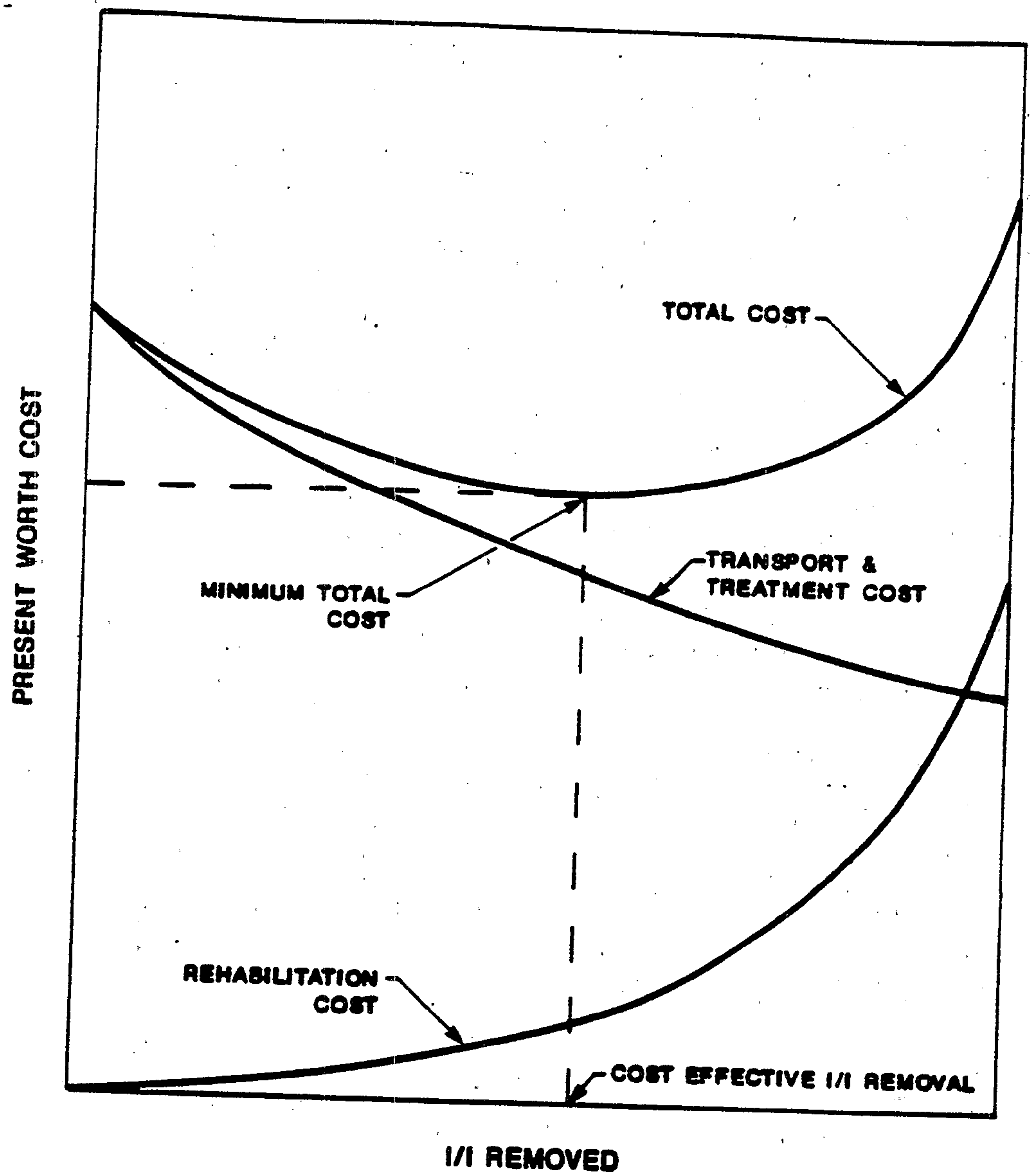


FIGURE 3-1

TRADITIONAL COST-EFFECTIVENESS CURVE

Control Methods

As noted previously, in sewer systems where the primary entry points of RII are defects in sewer pipes and laterals and where such defects are generally widespread, migration of RII to unrepaired defects may occur when only some points of RII entry have been eliminated. Additional defects may become active as the water seeks new entry points to the sewers. This migration factor has resulted in the failure of the traditional cost-effectiveness analysis approach to accurately predict the amount of extraneous flow reduction achievable through implementation of many correction programs originally calculated to be cost effective.

For this study, a cost evaluation was conducted to analyze the relative cost effectiveness of different rehabilitation approaches. Cost-effectivenss analyses were conducted for different "model" sewer systems, which were defined in terms of their age and general physical condition, magnitude and distribution of RII, and density of building laterals. The models were developed to evaluate sewer systems where the primary entry points of RII are defects in sewer mains and laterals, as opposed to systems in which the primary RII entry is through manhole frame/chimney defects, foundation drains, or other entry points not generally classified as pipe defects.

The purpose of the model system cost evaluation was to identify how the cost-effectiveness of RII correction was affected by the characteristics of the sewer system and by the type of rehabilitation approach selected. The rehabilitation approaches evaluated included:

- Isolated repair (spot repair of individual defects or specific pipe reaches).
- Rehabilitation of public sewer mains only.
- Rehabilitation of sewer mains plus lower laterals (the portion within the public right-of-way).
- Rehabilitation of sewer mains plus entire building laterals.

The cost analysis was designed to overcome the major limitation of the traditional cost-effectiveness methodology, that of overestimating rehabilitation effectiveness by ignoring the effects of flow migration. Two key assumptions were made:

- Rehabilitation was assumed to be conducted throughout contiguous areas within sewer subsystems, rather than only addressing individual RII entry points. To address a significant portion (at least 50 percent) of the RII in a subsystem, an area that included at least 30 percent or more of the "worst" sewers in the subsystem would require rehabilitation. The RII distribution within any particular subsystem could be assumed to fall within a generalized envelope, as shown in Figure 3-2.

- The assumed RII reductions (removable percentages of RII) assigned to various rehabilitation approaches were based on recognized limitations of "incomplete" system rehabilitation due to flow migration effects. Thus, large reductions (greater than 50 percent) were considered achievable only if the rehabilitation program included both the mains and laterals.

A detailed description of the assumptions and procedures used for the model system cost evaluation is presented in Appendix F.

The general results of the model system cost evaluation indicated that RII correction would probably not generally be cost effective in a "typical" old sewer system (sewers in generally poor condition and defects widespread) because of the high cost and need for extensive rehabilitation. However, under certain conditions (for example, a newer system with very high RII flows but low lateral density), RII could be cost effective if the mains and entire laterals were rehabilitated.

However, since the cost evaluation was applied to fictitious sewer systems and involved a number of assumptions regarding sewer system conditions and existing transport and treatment capacities, it was not intended to develop costs for RII control that could be applied to all sewer systems or draw definitive conclusions about the cost-effectiveness of RII correction in any specific sewer system. As noted previously, the types of RII correction programs addressed in the cost analysis are primarily aimed at correcting sewer system defects (RII entry points) using commonly applied techniques. Therefore, the cost evaluation did not consider the potential for RII reduction through improved design and construction standards, new or less costly techniques, or through as yet undiscovered methods that might be developed to intercept or divert water away from the pathways through which it reaches the sewers.

The cost effectiveness of RII correction is highly dependent on the capacity of existing downstream transport and treatment facilities and on the costs to provide any additionally required transport and treatment facilities. In a system where pipeline construction might be required in congested areas or under adverse soil or groundwater conditions, transport costs would be higher and RII correction could be more cost effective. Similarly, if treatment plant site constraints make overall plant expansion or construction of flow equalization facilities prohibitively expensive, the cost

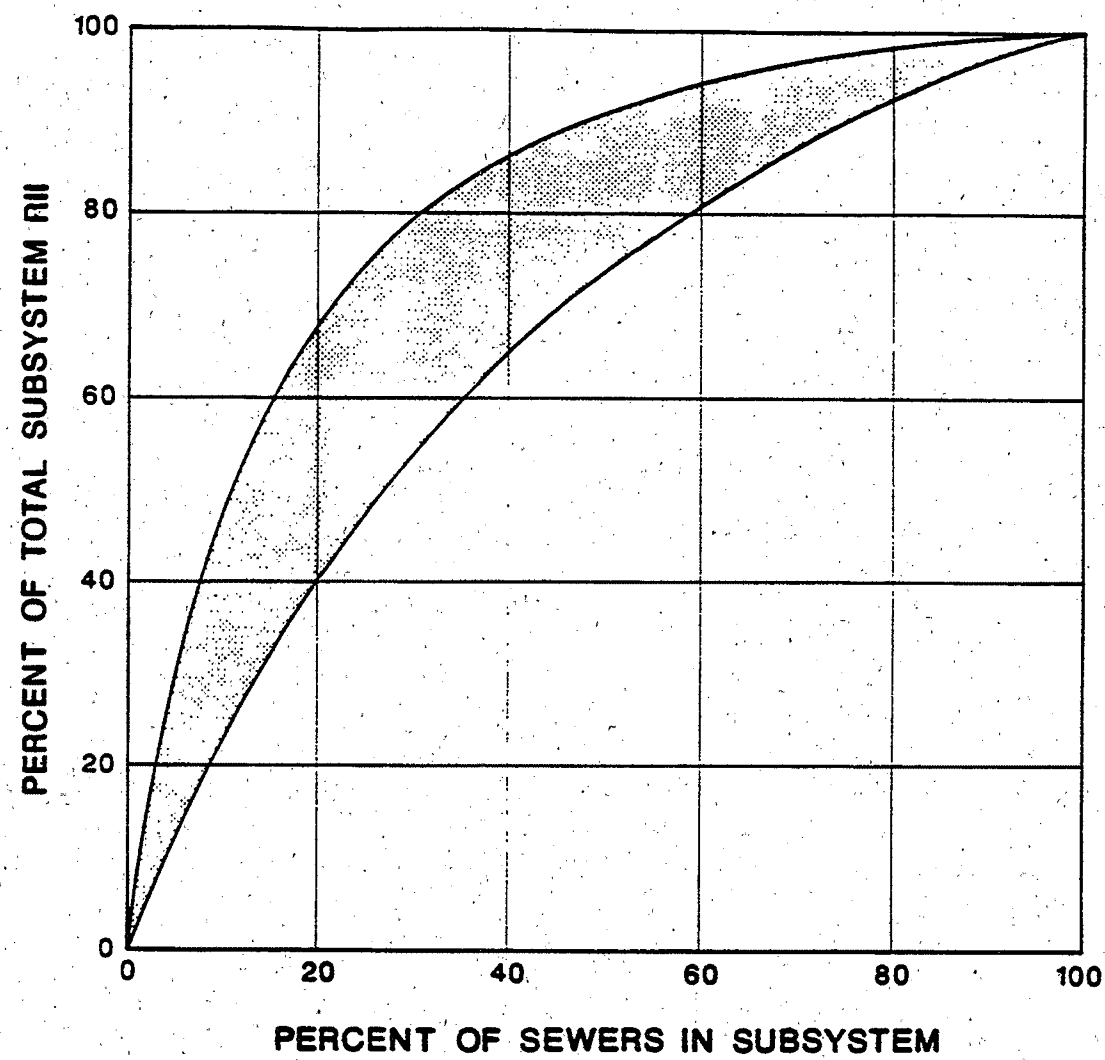


FIGURE 3-2

HYPOTHETICAL RII DISTRIBUTION

effectiveness of RII correction would also be higher. In the EBMUD system, for example, the high costs to transport and treat the peak RII flows have made RII correction cost effective in about 50 percent of the system. This is despite the fact that EBMUD is an old system with a relatively high lateral density.

An important point needs to made about old sewer systems with respect to assessing the cost effectiveness of rehabilitation. Many old systems have experienced significant structural deterioration. Good infrastructure management dictates rehabilitation, if only for the purpose of maintaining the structural integrity and proper functioning of the system. If it is recognized that life-cycle replacement and rehabilitation are integral parts of sewer system management, then the cost effectiveness of system rehabilitation can be assessed in terms of the benefits of both structural maintenance and RII reduction. In these cases, sewer system rehabilitation may be cost effective for reasons other than for RII reduction alone.

INSTITUTIONAL AND REGULATORY APPROACHES

Institutional and regulatory approaches can help facilitate implementation of rehabilitation programs and long-term RII control programs. These types of measures are particularly suited for RII control on private property.

Rehabilitation Programs

Institutional and regulatory measures that can be used in conjunction with rehabilitation programs include:

- Public agency ownership of laterals and/or responsibility for lateral construction.
- Financing programs (for public and/or private facilities).
- Enforcement (for private property rehabilitation).
- Public information programs.

Rehabilitation of both the public and private portions of a sewer system as part of a single, integrated construction project has distinct advantages in terms of lower cost, better quality control, and minimizing disruption to the community. One option available to an agency that is contemplating rehabilitation of sewers and laterals is taking over temporary ownership of the laterals during construction and assuming responsibility for maintenance of laterals for a one or two year warranty period after rehabilitation. These steps would allow the agency to perform any needed testing and rehabilitation without repeated contact with the property owner. After the agreed upon time period, the responsibility for the lateral would automatically revert to the property owner. The length of the time period selected should allow for completion of all necessary work and include a warranty period to ensure that the work has been properly undertaken.

Rehabilitation programs are expensive and may present considerable strain on the financial resources of both the public agency and the individual property owners. Financing options for public agencies include pay-as-you-go financing from general sewer use revenues, revenue bonds (repaid out of user fees), assessment district financing, and combinations thereof. Financing assistance may also be offered to individual property owners in the form of low-interest loans; local assessment district financing; or reduced costs through agency assistance in design of private property improvements, preparation of bid documents, and construction inspection.

Strict enforcement of requirements for private property repairs is another option. Municipalities with foundation drain connections to the sanitary sewer system have, in some cases, instituted inspection programs with mandatory removal of connected drains. Local ordinances have been passed for not disconnecting the drains, with penalties ranging from warnings to fines to forced disconnection. Similar regulatory methods can be used for enforcing building lateral rehabilitation. Such enforcement would require an ordinance that requires a building owner to maintain a properly operating lateral that does not contribute excessive non-wastewater flows to the sewer system, or require that the lateral be capable of passing a standard leakage test.

The success of an overall RII control program may greatly depend upon how well-informed the general public is regarding the need and requirements for the program. An effective public education program can potentially elicit substantial voluntary participation from individual property owners. For example, in Ames, Iowa, a public information program, combined with limited financial assistance, was successful in implementing necessary foundation drain disconnections on a voluntary basis.

Long-term Programs

Since new sewer facilities are often constructed by the private sector rather than a public agency, regulations provide a means of enforcing desired design standards and construction practices to minimize future RII. Preventive maintenance programs, particularly for private facilities, can be facilitated through regulations with requirements for testing, inspection, and repair of sewer system components found to be contributing RII.

Control Methods

One type of inspection program gaining in popularity in many parts of the country is required pressure or leak testing of private sanitary laterals at the sale of property. The test is required if the lateral has not been tested, reconstructed, or newly constructed within the previous 5 to 25 years, depending upon the municipality. While the obvious limitation of long-term property ownership will prevent all laterals in an area from being inspected under this type of program, the overall effect will be that a majority of laterals will be inspected over a reasonable time cycle. Integrating the testing requirement to the sale of property minimizes both the administrative burden on the agency and the financial impact on the property owner.

Lateral testing or inspection is dependent on the accessibility of the lateral. Cleanouts located at the curb and at the structure may be needed to effectively clean, inspect, or test each lateral. Consequently, some communities require cleanouts at both locations. In some localities where a curb cleanout does not exist, the agency will not maintain the lower portion of the lateral unless the owner installs one. In other municipalities where lateral testing or inspection is required, cleanouts are installed as part of the inspection process if they cannot be found. In some areas of the country where cleanouts outside of the building may not be practical because of weather conditions, provision for lateral access from within the house may be necessary to complete testing or inspection.

EXAMPLE RII CONTROL PROGRAM - EBMUD

This section describes an example RII control program, that of the East Bay Municipal Utility District in California. The problems associated with RII in EBMUD and a summary of the characteristics and documentation of RII in that system were presented in Chapter 2. This section specifically addresses the methodology used to quantify and identify locations of excessive RII in the sewer system, analyze the cost effectiveness of RII correction, and develop a comprehensive program to control RII in the EBMUD system.

The RII control program developed for EBMUD resulted from a six-year study, which culminated in the completion of SSES reports for the seven EBMUD communities. The basic objective of the study was to develop a cost-effective plan to solve the wet weather problems in the system due to excessive I/I. During the first year of the study, EBMUD and all of the communities and their respective consultants jointly developed a basic study methodology to guide the field investigations and data analyses to be conducted during the SSESs. While the fundamental concept of cost effectiveness was consistent with basic EPA requirements, a very detailed and rigorous approach was developed to quantify and identify I/I components and analyze the costs for I/I correction.

Field Investigations

The EBMUD sewer system consists of about 1,500 miles of sewer mains. The initial field effort involved focussing the study on those areas with the most significant I/I flows and therefore the greatest potential for cost-effective I/I correction. This was accomplished through a "gross" monitoring program, in which over 300 flow monitors, each with a tributary area containing an average of about 20,000 feet of sewer mains, were used to record sewer flows over a two-month period during the rainy season. For each monitor, storm flow data was "decomposed" into base flow and rain-induced I/I components. A method was developed to quantify the rain-induced I/I as a percentage of rainfall volume and describe the shape of the hydrograph with mathematical parameters. These parameters were then used to develop a "projected" hydrograph for each area of the sewer system for an established five-year design storm. This procedure enabled comparison of the rain-induced I/I flows from all areas of the sewer system on the basis of a common rainfall event.

Based on the results of the gross monitoring program, specific areas (subbasins) of the sewer system with the greatest potential for cost-effective I/I reduction were identified for further field work. The field work included smoke testing, dye flooding, manhole inspection, and flow isolation. Based on the results of flow isolation, specific sewer reaches within each subbasin were identified for TV inspection. As described in Chapter 2, the smoke testing results indicated that there were very few direct inflow sources in the system, and that laterals were a major potential contributor of RII. Other special field studies on laterals (TV inspection, leakage testing, rainfall simulation, and flow measurement during rainfall) were conducted to verify this hypothesis. TV inspection of sewer mains indicated that the sewers in the system were in generally very poor structural condition, with numerous cracks, offset joints, and other defects that serve as entry points for RII.

Cost-Effectiveness Analysis

The data obtained from flow monitoring and field testing and inspection were used to conduct a detailed analysis of the cost-effectiveness of sewer system rehabilitation to reduce I/I. Although the analysis addressed <u>all</u> components of I/I (SWI, RII, and GWI), the primary emphasis was on reduction of peak flows by controlling RII entry into the system.

Control Methods

The cost effectiveness of rehabilitation was calculated individually for each subbasin, based on the measured RII flows (projected to design storm conditions), RII flow distribution within the subbasin (from flow isolation), actual pipe footage and number of laterals, most appropriate rehabilitation method based on the physical conditions in the subbasin, and allocated share of downstream transport and treatment facilities. For a typical subbasin, 50 percent of the subbasin (contributing 80 percent of the RII) was assumed to be rehabilitated by slip-lining of the sewer mains and entire laterals, with an anticipated 70 percent reduction in RII over the 20-year analysis period. The results of the cost-effectiveness analysis indicated that rehabilitation would be cost effective in approximately half of the subbasins in the system. The remainder of the wet weather flows not eliminated by rehabilitation would be handled by new relief trunk sewers and District wet weather storage and treatment facilities.

Rehabilitation Program

Because the rehabilitation and relief sewer construction effort represented a major financial burden to the EBMUD communities, a 20-year construction program was developed. Projects were prioritized so that those with the highest cost effectiveness and/or the greatest benefit in terms of elimination of overflows would be constructed first. The program is currently in its second year of implementation. However, lack of rainfall has prevented the collection of post-rehabilitation flow monitoring data to assess the RII reductions achieved.

The rehabilitation methods used in the initial projects have included pipe replacement, slip-lining, inversion lining, and grouting. Because of the poor physical condition of most of the sewers, replacement and slip-lining are the most common methods used. In addition, a substantial portion of the existing system consists of substandard size six-inch diameter pipe, which dictates replacement with minimum eight-inch diameter pipe in most cases. Note that pipe replacement is needed primarily for structural and maintenance reasons, rather than strictly for RII correction. Therefore, the incremental cost of replacement over slip-lining more appropriately represents the cost to maintain the structural integrity of system, rather than a cost to reduce RII.

Although the recommended RII correction program includes both the public and private portions of the system, most of the initial rehabilitation projects have included only the mains and the lower laterals. Most of the communities recognize that implementing upper lateral rehabilitation involves institutional decisions that will require further study and discussion. In one of the communities, the initial rehabilitation project did include construction of the upper laterals as part of the public project. In the other communities, two-way cleanouts were installed at the property line to facilitate maintenance and future testing and inspection of the upper laterals. Options being considered for implementing upper lateral rehabilitation include low-interest loan programs and assessment district financing.

Long-term RII Control

In addition to the 20-year rehabilitation program, the EBMUD SSES reports recommended that the communities implement preventive maintenance programs for long-term RII control and adopt improved design and construction standards for rehabilitation and new sewer construction. The recommended preventive maintenance program includes flow monitoring of each major drainage basin every ten years to check for any significant wet weather flow increases; systematic testing and inspection of the sewer system on an approximate ten-year cycle; rehabilitation or replacement of sewers as indicated by testing and inspection; and routine cleaning and root control. A computerized maintenance management system was developed to facilitate this program.

SUMMARY

- Accurate field investigations and data analyses are important for developing an effective RII control program.

- Traditional I/I field data collection techniques can be used successfully to investigate and quantify RII in sanitary sewer systems if the techniques are properly applied and the data correctly interpreted.

- Many different methods are applicable for rehabilitation of sewer systems to reduce RII.

  Pipeline rehabilitation methods include in-place techniques, such as grouting and lining, as well as replacement by excavation or trenchless installation methods.

  Manhole rehabilitation techniques include both interior and exterior repair methods, many of which are specifically designed to address frame/chimney defects.

- Most pipe rehabilitation techniques can be used for both sewer mains and building laterals, but are more costly and difficult to apply to laterals.

- System repair for RII control on private property has generally not been done because of financial and institutional constraints and the difficulties of access.

Control Methods

- A comprehensive program of sewer system rehabilitation that includes both the public and private portions of the system can be effective in reducing RII.

- Modified design standards and construction practices can provide a means to minimize future RII into new or rehabilitated sewers by preventing the development of sewer defects and minimizing the migration of extraneous water to defects.

- An effective sewer system preventive maintenance program can provide a means for long-term control of RII.

- The "traditional" cost-effectiveness analysis approach may overestimate the amount of flow reduction achievable through I/I correction because it fails to account for the migration of RII to unrepaired defects.

- The cost effectiveness of rehabilitation to control RII is specific to each sewer system.

  Cost effectiveness depends upon the physical conditions in the system, magnitude of RII, capacities of existing transport and treatment facilities, and the relative costs to construct additional capacity.

- Institutional and regulatory approaches may be necessary to effectively control RII in order to provide for enforcement, financing, and public information for private property rehabilitation.

RECOMMENDATIONS

- The specific analysis of RII should be included as part of overall I/I evaluations.

- Guidelines should be developed to ensure the proper application of field techniques and interpretation of data to identify and evaluate RII.

- Direct inflow sources should be identified and removed first so that a correct assessment can be made of the need for and potential flow reductions from RII control.

- Appropriate rehabilitation techniques should be selected based on site specific conditions, such as existing physical conditions and types of defects (entry points of RII).

Control Methods

- The following considerations should be incorporated into the development of sewer system rehabilitation programs and evaluation of the cost effectiveness of rehabilitation:
  - Addressing entire areas of the sewer system versus repair of individual defects only.
  - Including both the public and private portions of the sewer system versus only the public portion.
- Long-term RII control should be ensured through implementation of an effective preventive maintenance program that includes:
  - Periodic flow monitoring in the system to identify areas with increases in RII levels.
  - A routine program of cleaning and root removal.
  - A cyclic program of testing and inspection of the sewers throughout the system to identify the need for repairs and replacement.
- Sewer design standards should be modified to include the following considerations:
  - Restricting the flow of water in granular backfill.
  - Reduction of utility trench backfill interconnections.
  - Control of migration of fine soil or backfill material particles.
  - Reduction in the number of pipe joints.
  - Incorporation of pipe system flexibility to reduce settlement stresses.
  - Improved sealing of pipe connections at manholes.
  - Provision for tight, but flexible, lateral connections.
  - Provision for access for testing, inspection, and repair of laterals.
- Effective sewer construction practices should be followed to ensure that design standards are met by:

- Regular construction inspection.

- Adequate performance testing for public sewer mains as well as private laterals.

o The institutional and regulatory framework governing the construction and maintenance of house laterals (the connection between the house or building and the collector sewer in the street or other public right-of-way) should be re-examined. Possible options include:

  - Shifting responsibility for construction and/or maintenance of house laterals from the home owners to the municipality.

  - Municipal programs to help home owners pay for maintenance and repairs of house laterals.

  - State or municipal ordinances, with appropriate enforcement provisions, governing inspection, testing and repair of house laterals.

  - Public education programs to inform citizens of the importance of excluding extraneous flows from the municipal sanitary sewerage systems.

o The development of new, improved, and potentially less costly sewer rehabilitation techniques, particularly for laterals, should be encouraged.

o The collection and publication of data documenting the effectiveness of different rehabilitation methods and approaches to controlling RII should be encouraged.

APPENDIX A

LIST OF ABBREVIATIONS

ABS	Acrylonitrile butadiene styrene pipe
AC	Asbestos cement pipe
ADWF	Average dry weather flow
BWF	Base wastewater flow
BOD	Biochemical oxygen demand
C-E Ratio	Cost-effectiveness ratio
CMP	Corrugated metal pipe
CSO	Combined sewer overflow
EBMUD	East Bay Municipal Utility District
EPA	U.S. Environmental Protection Agency
gpcd	Gallons per capita per day
gpd	Gallons per day
gpm	Gallons per minute
GWI	Groundwater infiltration
I/I	Infiltration/inflow
mgd	Million gallons per day
mg/l	Milligrams per liter
MMSD	Milwaukee Metropolitan Sewerage District
NEORSD	Northeast Ohio Regional Sewer District
NPDES	National Pollutant Discharge Elimination System
O&M	Operation and Maintenance
PVC	Polyvinyl chloride pipe
PWWF	Peak wet weather flow
RDI/I	Rainfall dependent infiltration/inflow (same as RII/I)
RII	Rainfall induced infiltration
RII/I	Rainfall induced infiltration/inflow

SSES	Sewer system evaluation survey
SWI	Storm water inflow
TSS	Total suspended solids
VCP	Vitrified clay pipe
WWTP	Wastewater treatment plant

APPENDIX B

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APPENDIX C

CASE STUDIES

This appendix contains detailed descriptions of the RII case studies summarized in Chapter 2. The case studies are:

- o East Bay Municipal Utility District, California
- o City of Springfield, Oregon
- o Milwaukee Metropolitan Sewerage District, Wisconsin
- o Northeast Ohio Regional Sewer District, Ohio
- o City of Baton Rouge, Louisiana
- o City of Springfield, Missouri
- o North and South Shenango Joint Municipal Authority, Pennsylvania
- o City of Ames, Iowa
- o City of Coos Bay, Oregon
- o City of Tulsa, Oklahoma

EAST BAY MUNICIPAL UTILITY DISTRICT, CALIFORNIA

The EBMUD wastewater service area is located in northern California on the eastern shore of San Francisco Bay. It includes seven community wastewater collection agencies. EBMUD operates the interceptor system and treatment facilities which transport and treat the wastewater generated from these seven communities. The collection systems, which include about 1,500 miles of sewer main, are owned and operated by the individual communities. Although the original sewers installed prior to 1938 were constructed as combined storm drainage and sanitary sewage facilities, the systems are now entirely separate sanitary systems.

The community collection systems, as well as the EBMUD interceptor and treatment facilities, do not have adequate capacity to handle the peak flows which occur during wet weather. As a result, overflows onto city streets and bypasses to local watercourses and to San Francisco Bay occur at numerous locations within the community systems and at seven locations along the EBMUD interceptor. Peak wet weather flow rates may exceed twenty times the average dry weather flow in the system.

In 1980, the wet weather problems in the EBMUD service area led to the initiation of the East Bay Infiltration/Inflow Study to address the problems within the community collection systems. Concurrent with the East Bay I/I Study, EBMUD conducted a wet weather facilities plan for its interceptor and treatment facilities. The East Bay I/I Study included extensive flow monitoring and SSES field work within the community collection systems. One of the major conclusions of the study was that the major portion of the peak wet weather flows in the EBMUD system are due to infiltration of storm water into defective pipes and manholes. This RII appeared to exhibit similar flow characteristics as direct storm water inflow, with very rapid, high peak flows occurring in direct response to rain storms. A major source of the RII is believed to be defective building laterals.

System Description

The EBMUD wastewater service area is located on the east shore of San Francisco Bay, extending eastward to the steep hills that form the eastern and northern boundaries of the area. Most of the developed portion of the service area is located on an alluvial plain, at an average elevation of 75 feet and a width of from one to three miles, which rises gently from the Bay shoreline eastward to the foot of the hills. Predominantly newer development is located in the hill areas, which rise to an average crest of 1,200 feet.

Rainfall. The average annual rainfall, as measured at the Oakland Airport located on the shoreline of the Bay, is about 18.7 inches, with 90 percent of the rain falling during the period November through April. Winter storms move through the area from west to east, generally depositing a greater amount of precipitation in the higher elevations of the study area. (Hence, the actual average rainfall in the study area is higher than the Oakland Airport data.) During the winter season, storms may occur one after another during extended rainy periods, or dry periods of up to several weeks without any rain may occur.

Soils. The soils within the study area range from loose sediments, such as bay muds in the marshy tidal flats, to sedimentary rocks in the hillsides. The tidal flats, consisting of clays and silty clays, extend along the perimeter of the Bay. At higher elevations, soils were formed along flood plains, river-mouth fans, and low terraces, and include silty clays, clays, silty clay loams, clay loams, and sandy clay loams. Much of the soils in the EBMUD service area have a high shrink-swell potential and low percolation rates. Under prolonged dry periods, as occur during the summer months, the soils are subject to shrinking and cracking.

Hydrogeology. Numerous small streams discharging to the Bay drain the basins formed by the Oakland-Berkeley hills. These streams, together with the intervening ridges, form the topographic features that describe tributary areas (basins) of the wastewater collection systems. Groundwater levels in the study area range from less than five feet below the ground surface in locations near the Bay shoreline to greater than 10 feet below the surface in the hill areas. However, some higher groundwater levels may be found in localized portions of the higher topographic areas due to in-filled stream channels or soil variations in the Hayward fault zone, which forms a north-south band through the foothills. Groundwater levels in the study area generally vary on a seasonal basis, with the lowest levels occurring in early fall after the long dry season, and the highest levels in the spring at the end of the rainy season.

Sewer System. The first sewers in the EBMUD area were constructed in the 1880's. The original sewers were six- and eight-inch diameter clay pipes which served as a combined storm/sanitary system and generally discharged the flow to the nearest drainage channel. Most of the early trunk sewers were enlarged and extended during the 1920's and 1930's, with downstream discharges near the Bay shoreline. In 1951, the EBMUD interceptor system was constructed along the Bay shoreline to intercept the east-west community trunk sewers and convey the flow to the new treatment plant.

The major portions of the existing EBMUD community sewer systems were constructed in the first part of this century, and consist primarily of vitrified clay pipe (VCP) with short, two- or three-foot pipe sections and rigid, cement mortar joints. Most of the early sewers were laid with bedding and backfill composed of the native soil materials. Soil logs from groundwater monitoring wells drilled adjacent to sewer pipes for the East Bay I/I Study show that the boundary between the trench fill and the native soils beneath the trench is generally indistinguishable. Most of the sewer system was constructed piecemeal by individual developers with little, if any, construction inspection or quality control provided by the cities. In addition, maintenance of the sewer system over the years has been minimal, other than that required for emergency situations such as blockages or street collapse. Because the sloping topography of the area facilitates gravity flow from east to west, and because most building laterals are shallow (homes generally do not have basements), most of the sewers are relatively shallow (typically four to six feet deep), with deeper pipes being necessary only for the larger downstream trunk sewers nearer the interceptor. The sloping topography also means that travel times through the sewer system are short, and upstream peak flows cumulate rapidly and reach the downstream end of sewer drainage basins in a relatively short period of time (typically, within one to two hours or less).

The study area is characterized primarily by urban, single-family residential development on small lots. Therefore, the density of building laterals is relatively high, with an average of 22 laterals per 1,000 feet of sewer main. Many of the original laterals were not connected to the factory-installed wye fittings in the sewer main, but were inserted through holes (taps) broken or chipped into the pipe. In these cases, some reaches may have as many inactive and unplugged factory wyes as active lateral connections. The upper portions of building laterals on private property are generally very shallow (less than three feet deep), with a change in vertical alignment typically occurring at the curb line where the pipe angles down toward the main sewer in the street.

RII Documentation. The condition of the EBMUD sewer systems has been documented by the field work conducted as part of the East Bay I/I Study. The field work included smoke testing, dye flooding, physical inspection of manholes, internal television inspection of sewer mains, and lateral testing and inspection.

Smoke testing was conducted in over 50 percent of the EBMUD system. The majority of the smoke returns were from defective building laterals. Direct storm water inflow connections accounted for relatively few of the observed defects.

Dye flooding was conducted to verify suspected storm drain/sanitary sewer cross connections detected during smoke testing or potential "indirect" connections where storm drains crossed over or closely paralleled sanitary sewers. In most of the dye flooding tests, the sanitary sewer was concurrently TV inspected in order to observe the exact location and relative quantity of dye transfer. With only a very few exceptions, most of the instances of flow transfer from the storm to the sanitary sewers were found to be cases of indirect transfer via exfiltration of water out of the storm drain and infiltration into the sanitary sewer through cracks and defective joints.

Manhole inspection was conducted for about 20 percent of the structures in the system. In general, the inspections indicated that most manholes were in good structural condition with relatively little evidence of infiltration. Based on these inspections, it was concluded that manholes were not a significant source of RII in the EBMUD system.

TV inspection was conducted for about 10 percent of the total sewer main footage in the service area, based on I/I flow contribution as determined through flow monitoring and flow isolation. Numerous defects in the system were identified through TV inspection, including structural problems, cracks, offset joints, root intrusion, and defective lateral connections. The TV inspection results were used to document the condition of the pipes in the system and determine appropriate rehabilitation methods, but were not used to attempt to quantify the I/I contribution from individual sources.

Lateral Testing and Inspection. Lateral field work conducted as part of the East Bay I/I Study included air and exfiltration testing, rainfall simulation, flow measurement during rainfall, TV inspection, and visual inspection of exposed laterals. Most of this work was done as part of special pilot projects.

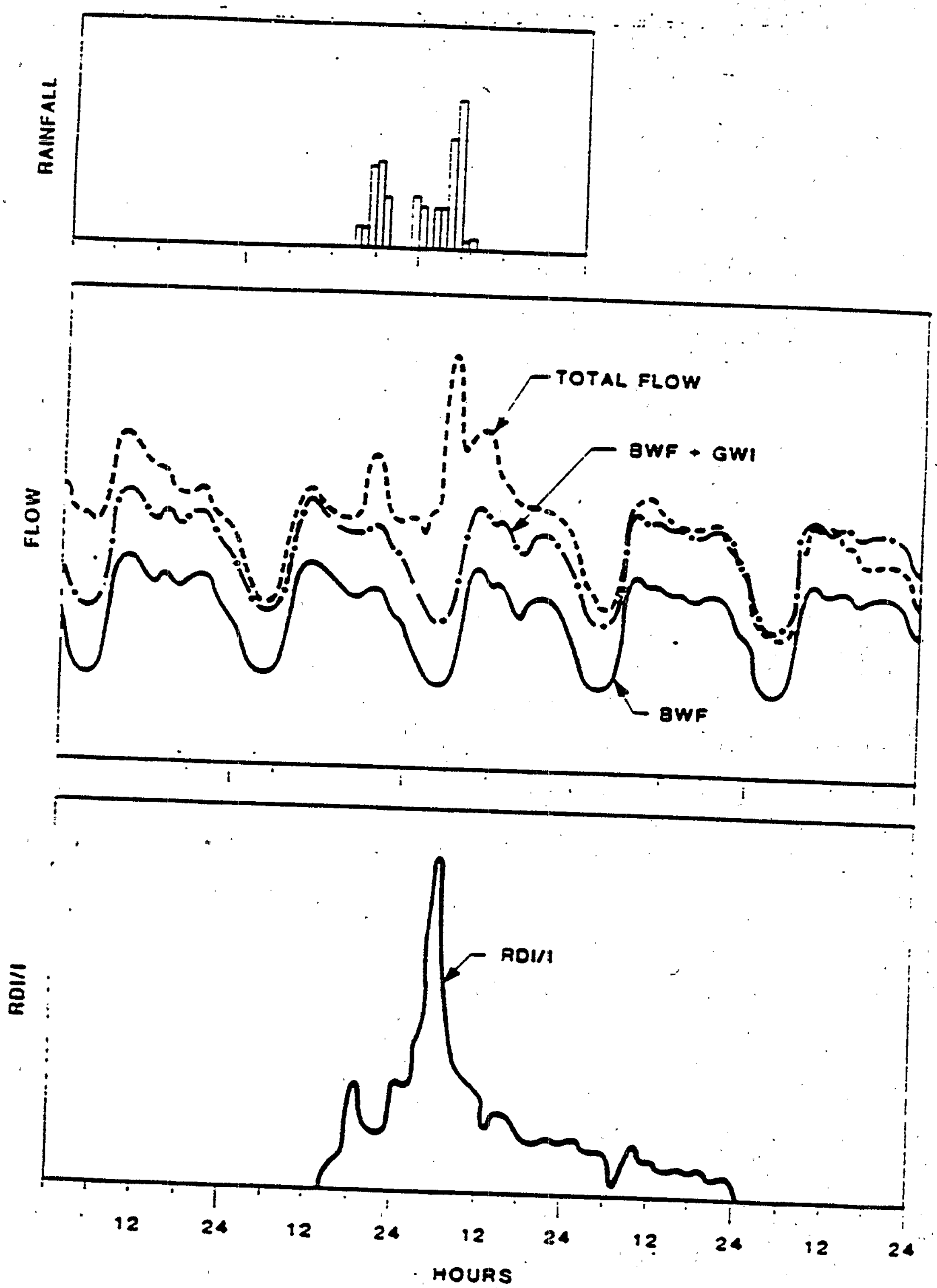
The lateral field studies generally included samples ranging from 10 to 200 laterals. While these samples represent a small fraction of the total 175,000 laterals in the EBMUD service area, the areas were selected to be representative of typical conditions in the area. Lateral inspections revealed that offset joints and root intrusion occur in most laterals, and other defects such as cracks (particularly near the bells of the pipes) and misalignment are common. A limited program in which eleven lower laterals (portion within the public right-of-way) were excavated and exposed using "archeological" methods showed that in 90 percent of the laterals, the original mortar in the joints had deteriorated. Most laterals failed air and exfiltration tests, and the ones that passed were generally newer pipes or atypical (e.g. cast iron rather than VCP construction). A comparison of smoke testing records with the results of other lateral testing and inspection methods indicated that only about one-third of defective laterals were detected by smoke testing.

I/I Flow Characteristics. The East Bay I/I Study included extensive flow monitoring within the community sewer systems. Fifty-six "long-term" flow monitors were installed for a period of two to three years, and wet weather flow monitoring was conducted in over 300 locations in the system for periods of approximately two to three months during the rainy season.

The analysis of flow data for the study was based on separating the total wet weather flow into its component parts of base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent I/I (RDI/I). The RDI/I was assumed to represent a combination of direct stormwater inflow (SWI) and rainfall-dependent or rainfall induced infiltration (RDI or RII); however, it was recognized that the SWI and RII components could not necessarily be distinguished by simple flow hydrograph decomposition. The combination of BWF plus GWI was determined based on the flow during a non-rainfall period near the time of the storm event being analyzed. Subtraction of BWF + GWI from the total storm flow hydrograph yielded the RDI/I for the rainfall event, as shown in Figure C-1. The RDI/I volume was then expressed in terms of a percentage of the total rainfall volume for the storm. This percentage, the ratio of the RDI/I volume to the total rainfall volume, was termed the "R value" or "total R" for the storm event. R values ranged from near zero in some subbasins to over 50 percent in others, depending on soil saturation (antecedent rainfall) and other factors.

Plots of rainfall volume versus RDI/I volume for all storm events were developed for each of the 56 long-term monitors. It was found that the plotted points fit within an "envelope" (see Figure C-2), with the storms representing early season or dry soil conditions (low R values) falling near the lower boundary of the envelope, and the storms representing saturated soil conditions (high R values) falling near the upper boundary. The dry soil R values were generally in the range of 2 to 6 percent. The saturated soil R values typically ranged from 10 to 35 percent for the long-term monitoring sites.The interpretation of the RDI/I envelope lower boundary condition is that it represents predominantly SWI contributions, since the percentage of runoff from impervious surfaces that collect surface drainage is more or less independent of antecendent rainfall conditions. The remaining RDI/I volume, and possibly also a portion of the RDI/I volume represented by the lower envelope boundary, is suspected of being contributed from infiltration sources such as defects in sewer mains and laterals, including indirect transfer from storm drains to sanitary sewers.

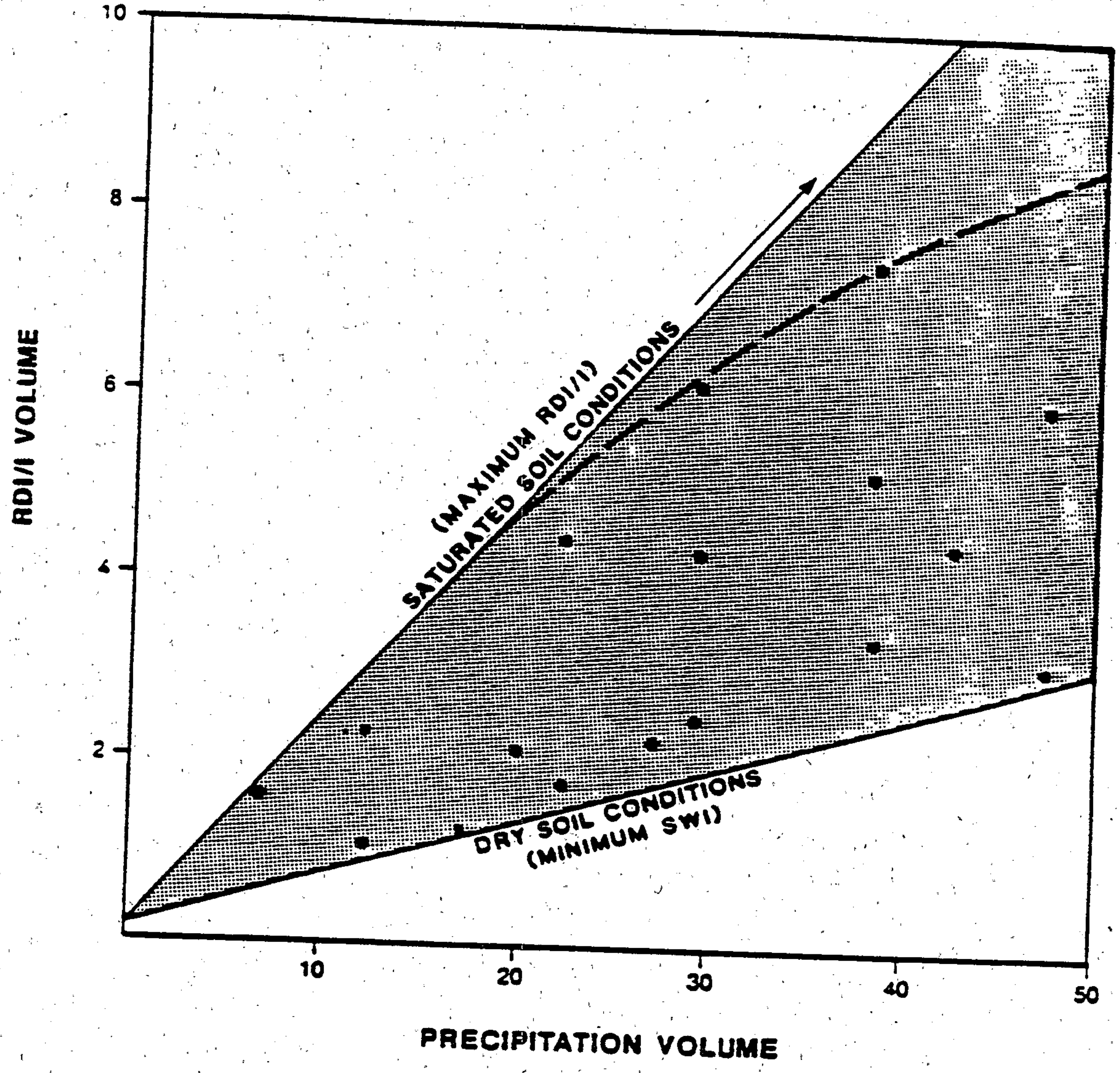
The interpretation of the upper envelope boundary is that it represents maximum RDI/I contribution under saturated soil conditions. Under such conditions, the capacity of the soil mantle to absorb and transmit water would be limited, and more water would be transmitted through soil channels and through the more permeable pipe trenches to sewer defects. In some cases, the saturated soil condition appeared to be better represented by a curvilinear upper boundary, indicating a reduction in R for larger storms because of the limitation in the amount of water that can reach the defects in the pipes once the soil has become saturated, as well as the inlet hydraulic capacity of the defects themselves. The average R value for the study area under saturated soil conditions (for the selected design rainfall event) was found to be approximately 18 percent. If the R value under dry soil conditions (average of about 4 percent) is assumed to be the maximum SWI, then this means that three-quarters or more of the total RDI/I volume under saturated soil conditions is due to infiltration of stormwater into the system, or RII.



EBMUD I/I STUDY

TYPICAL DECOMPOSED HYDROGRAPH

FIGURE C-1



= RAINFALL-DEPENDENT INFILTRATION (RII)

ALTERNATE CURVILINEAR UPPER BOUNDARY

EBMUD I/I STUDY

TYPICAL BASIN RDI/I ENVELOPE

FIGURE C-2

To more precisely define the magnitude and shape of the design storm hydrograph for use in modeling of the sewer system, the RDI/I hydrograph for each subbasin was separated into three component R values (R1, R2, and R3) which summed to the total maximum (saturated soil) R for the subbasin, as illustrated in Figure C-3. In general, the R1 component represented the most rapid response, with a time to peak of from one to two hours after the start of rain. The R1 component could therefore be assumed to represent SWI and a portion of RII, presumably from shallow infiltration sources. R1 was the dominant component in determining the magnitude of the peak storm flows.

To estimate the magnitude of SWI independently of the flow monitoring data, smoke testing data were used to identify specific SWI sources and develop quantitative estimates of the SWI contribution from those sources. The SWI contributions from all sources in each subbasin were then summed and compared to the design storm R1 volume for the subbasin. In almost all cases, the calculated SWI volume as a percentage of the R1 volume was less than 10 percent, and in most cases was less than 5 percent.

While the calculated SWI is probably an underestimate of the actual SWI volume because not all SWI sources may have been included, the estimated numbers did indicate that SWI appears to be only a small part of the peak (R1) component of RDI/I. Therefore, it was concluded that the major portion of the peak RDI/I flow in the EBMUD system is due to infiltration (RII), rather than inflow, into the sewers. Based on the high number of defective laterals detected during smoke testing, as well as the results of the lateral testing and inspection work, it was surmised that the peak RII flow is largely due to the rapid infiltration of stormwater into shallow, defective laterals. The magnitude of this RII flow can in great part be explained by the overall poor condition, as well as the high density of the laterals in the system.

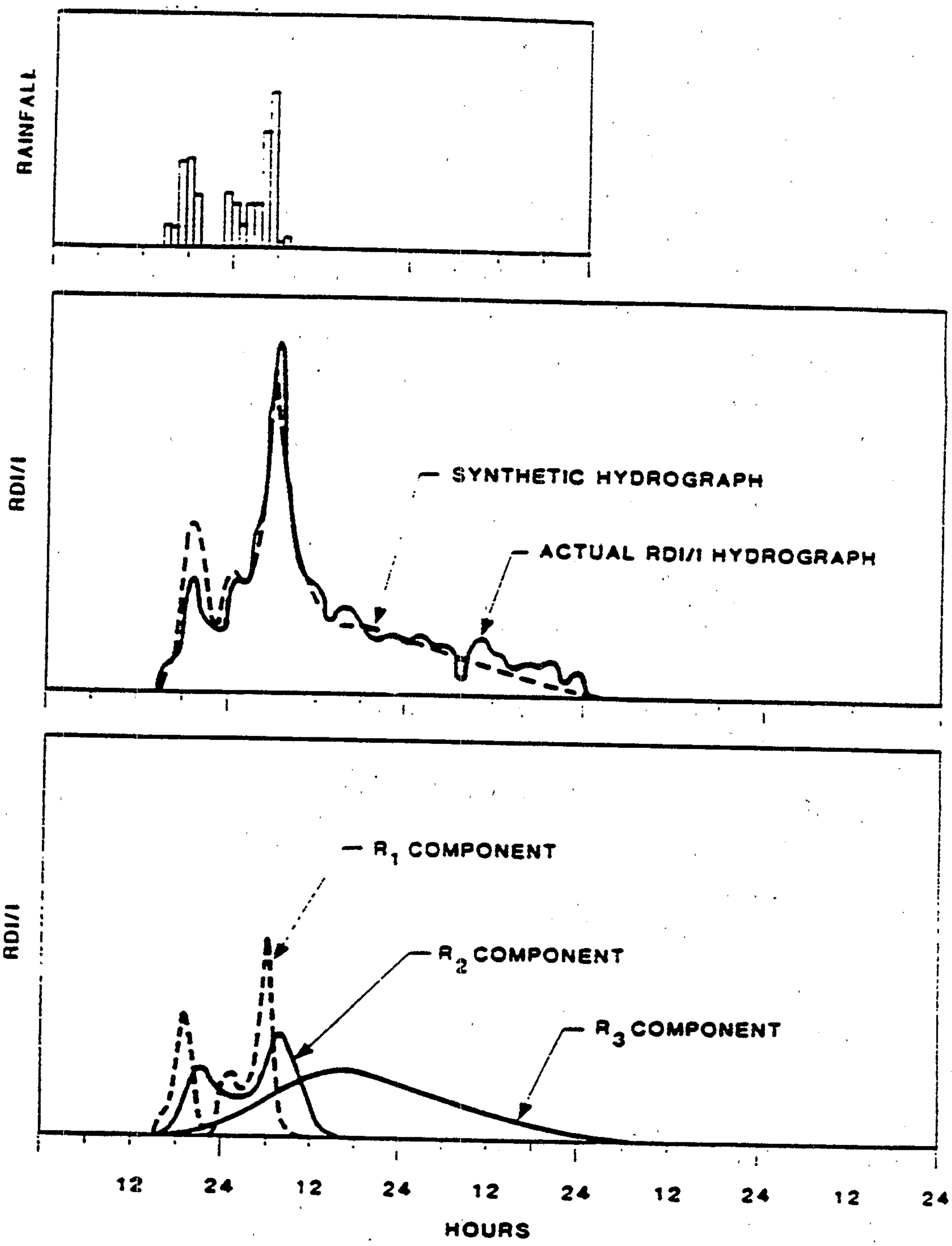
Several of the field studies that were conducted on laterals as part of the East Bay I/I Study appear to confirm the rapid flow response in laterals to rainfall events. These studies included actual flow measurement of laterals which discharged directly to manholes. During relatively low intensity storms (on the order of 0.1 inches per hour rainfall), approximately two-thirds of the laterals sampled contributed an average peak flow of 750 to 800 gpd per lateral. Some individual laterals contributed as high as 5,000 gpd peak flows. (Smoke testing records were used to verify that no direct SWI connections existed for these laterals.) When projected to a higher intensity design storm, the average peak flow contribution from laterals could be greater than 3,000 gpd per lateral. In most laterals contributing RII, the peak RII flow occurred within an hour of the peak rainfall intensity. Several laterals overlain by relatively impermeable surfaces also exhibited high peak flow responses, possibly indicating that infiltration can apparently move in a horizontal direction, as well as downward through the soil.

Rainfall simulation testing also confirmed the rapid flow response of laterals to rainfall. In this program, 230 laterals were tested by application of simulated rainfall (at a measured rate) in a six-foot wide spray zone over the upper portion of the lateral (portion upstream of the sidewalk). The resulting flow from the lateral was then measured from the sewer main using a weir/packer assembly attached to a TV camera. The flow hydrographs indicated a rapid response to the rainfall simulation, with the peak flow generally occurring within one to two hours after the start of the simulated rainfall. In a few laterals, the simulated rainfall application rate was increased after several hours of testing, and the flow response was an almost immediate increase in the measured infiltration rate. This response appears to indicate that infiltration rates are related to rainfall rates, and for a given lateral, an increase in rainfall intensity will cause an increase in infiltration.

The factors which impact RII flows in the EBMUD system include the physical characteristics of the service area, including clay soils, seasonal rainfall pattern, and sloping topography (which influences sewer depths and flow travel times), as well as the physical condition and characteristics of the sewer system. The age and original poor construction, type of pipe material (VCP with short pipe lengths and deteriorated cement mortar joints), lack of maintenance, relatively shallow depth (particularly of laterals), high density of sewers and laterals, occurence of root intrusion from landscaping, and pipe damage and joint separation caused by earth movement and seismic activity are all factors which have resulted in a large number of defects in the system through which infiltration can enter. The flow data collected as part of the East Bay I/I Study document the high peak RII flows which occur in the system.

RII Control Program

The analysis conducted for the East Bay I/I Study found that rehabilitation was cost effective for approximately one-half of the subbasins. The recommended I/I correction program consists of "comprehensive rehabilitation," i.e., including the sewer mains and the entire portion of the service laterals. Because of the high cost and size of the construction effort, the rehabilitation program will be implemented over a period of 20 years.



EBMUD I/I STUDY

TYPICAL SYNTHETIC HYDROGRAPH

FIGURE C-3

To date, the cities have completed design and construction of the first two years of projects. Most of the rehabilitation work has consisted of slip-lining and replacement, with some grouting. In most of the cities, only the sewer mains and the portion of the lower laterals within the public right-of-way have been constructed, and two-way cleanouts have been installed on the laterals at the property line. Private lateral rehabilitation will be addressed at a later date. However, one of the seven tributary agencies, the Stege Sanitary District, elected to construct and finance the rehabilitation work on private property in one subbasin.

In addition to the 20-year I/I correction program, it was recommended that the communities implement long-term I/I management programs. These programs would provide for routine testing, inspection, and maintenance of the sewer system and a cyclic replacement program for sewers that have outlived their useful lives.

CITY OF SPRINGFIELD, OREGON

The City of Springfield is located in central western Oregon at the confluence of the McKenzie and Willamette Rivers. The City's sanitary sewer system is tributary to a regional wastewater treatment plant serving the Cities of Eugene and Springfield. The City of Springfield system serves a population of about 40,000 and includes approximately 165 miles of sanitary sewer mains.

In the late 1970's a regional wastewater management study was conducted for the Eugene/Springfield area to identify appropriate means for expanding and upgrading the existing wastewater facilities. At that time, Eugene and Springfield were served by separate wastewater treatment plants. Problems in Springfield included surcharging and overflows in the sewer system and bypassing of partially treated wastewater from the Springfield treatment plant during wet weather periods (almost continuously during December and January). The recommended project included construction of a regional treatment plant (completed and in operation since 1984). As part of the facilities planning phase of that project, Springfield conducted an I/I Analysis, which determined that I/I was "excessive", and subsequently completed a SSES in 1980. The SSES determined that only 20 percent of the design peak storm induced flow could be attributed to direct inflow sources and indirect transfer from storm drains to sanitary sewers. Therefore it was concluded that 80 percent of the peak storm induced flow was due to "storm induced infiltration" through defective sewers, service laterals, and manholes.

System Description

Most of the service area is relatively flat, with a typical 100-foot east to west variation in elevation. As most of the City is located in a river floodplain, the soils are primarily alluvial deposits, ranging from gravelly silt-loams to silty clay-loams. The climate is typical of the west coast of the U.S., with about 80 percent of the total rainfall falling during the period November through April. Average annual rainfall is approximately 45 inches. The groundwater table is typically 10 to 20 feet below the ground surface during the summer, with about a seven foot annual fluctuation. The groundwater is highest during the winter rainy season, and very near the surface in the western portion of the City near the river confluence.

The sewer system was originally constructed in the central portion of the City between 1910 and 1940, with expansion of the system into the eastern and northern portions taking place since 1940. The older sewers are VCP or concrete with cement mortar or asphalt-poured joints. Many of the older service laterals were constructed of Orangeburg pipe, although many of these have presumably since been replaced. Newer construction since 1960 has been primarily concrete pipe with rubber gasket joints. The depth of sewer mains ranges from 5 to 11 feet, with an average depth of 8 to 9 feet. Groundwater monitoring conducted during the SSES indicates that a large portion of the sewer mains are below groundwater during the winter. There are approximately 13,000 service connections in the system, for an average lateral density of 15 per 1,000 feet of main.

RII Documentation

During the SSES, dry and wet weather flow monitoring was conducted at 54 sites throughout the sewer system. For each monitored area, the average dry weather flow, peak non-rainfall infiltration rate and peak storm induced I/I rate were determined from the flow data. For the measured storms, peak to average flow ratios ranged from about 1.5 to 15. The peak storm-induced flow was projected to a five-year design storm condition based on the ratio of measured (two-hour) rainfall to design rainfall intensity. For the total system, the ratio of design PWWF to ADWF was approximately 11 to 1.

The field investigations conducted as part of the SSES identified numerous defects and I/I sources in the system. Smoke testing was conducted for over 90 percent of the sewers. A large proportion of the smoke emissions were observed along the ground above sewer mains and laterals and near manholes. Other smoke emissions were from manhole frames and lids, cleanouts, and storm drainage sources (catch basins, storm sewer manholes, area and roof drains).

To verify whether storm drainage sources were direct or indirect connections and to quantify the flow contribution from these sources, dye flooding was conducted for all smoke emissions from catch basins, storm sewers, and area drains. The results of the dye flooding indicated that over 90 percent of these sources were cases of indirect flow transfer between storm and sanitary sewer facilities; only five direct connections were found. Physical reconnaissance was conducted for all other specific smoke emission sites, such as cleanouts, which appeared to be potential sources of direct inflow. In addition, all manholes in the system were inspected to identify potential inflow sources through holes in manhole covers.

Television inspection was used to identify specific defects in sewers where infiltration during dye flooding was identified. TV inspection or review of past TV inspection records was also conducted for those sewers where smoke was observed along the ground surface over the pipe.

For all direct inflow sources, estimates of maximum flow rate were made using the rational formula, based on the surface area and drainage characteristics of each source and the design rainfall intensity. Flow estimates based on dye transfer rate were made for indirect connections between storm drains and sanitary sewers. The calculated total peak flow from these sources was 13 mgd, or 20 percent of the projected peak storm induced I/I flow of 65 mgd. Since a portion of the 13 mgd is due to indirect transfer from storm to sanitary sewers, it can be concluded that over 80 percent of the peak storm induced I/I appears to be due to RII.

A site visit was made to Springfield during the course of this study, and City staff were interviewed regarding RII problems in the sewer system. With respect to the condition of the sewers, staff identified service laterals as potentially significant contributors of extraneous flows. Specific problem areas are the connections between the private and public portion of the lateral, between the lateral and the main, and at the manhole. Other potential causes of RII include inactive, unplugged lateral taps and root intrusion. Because many of the older mains are located in backyard alleys, unauthorized and uninspected hookups and repairs are frequently made. High flows are also experienced in newer areas built during the mid-1970's development "boom," when construction inspection may have been inadequate because of insufficient City staffing.

Some the primary factors affecting RII in Springfield, in addition to the condition of the sewers, appear to be the high groundwater and amount and pattern of rainfall. During storms, groundwater in the trench line has been known to wash out portions of streets and create small "geysers" up through the asphalt. The flow response to rainfall is rapid, and generally decreases rapidly after the end of rain. This rapid rise and recession appear to be independent of the groundwater infiltration rate immediately prior to the storm. Larger flows occur during prolonged rainfall periods than from isolated rain events.

RII Control Program

As a result of the SSES and further cost-effectiveness evaluations, the City received a construction grant for sewer rehabilitation in four basins, representing approximately five percent of the total system. (The official grantee is the Metropolitan Wastewater Management Commission, the regional wastewater agency for the Cities of Eugene and Springfield.) The area is an older part of the City, with most of the original sewers over 40 years in age and constructed predominantly of concrete pipe with cement mortar joints. This rehabilitation project (called the "C74" project) was based on a design philosophy of "complete basin" rehabilitation (all of the sewers, including service laterals) and was projected to result in a 65 percent I/I flow reduction. The first phase, consisting of replacement or grouting of the mains and service laterals within the public right-of-way, was completed in 1987; preliminary flow monitoring results indicate that an approximate 50 percent flow reduction has been attained.

In addition to the grant project, several small pilot projects have been completed, including two in the C74 area and two in newer areas of the City. The C74 pilot projects involved rehabilitation of private service laterals where the main had already been replaced or grouted. The other two projects were done under a turnkey-type contract in which the contract bid was based on achieving 65 percent flow reduction with a price incentive for greater reductions. Analysis of the flow reductions achieved in these pilot areas are not conclusive because of the lack of rain during the 1987/88 season. However, preliminary results of the two projects in the newer areas indicate 90 percent and over 50 percent flow reductions, respectively, based on total storm flow volume calculations.

The City is currently re-assessing its approach to addressing I/I and developing a long-term I/I control plan. In general, City staff feel that sewer system rehabilitation, particularly in older areas, is not cost effective. The City is looking at other options, including off- line storage for peak flows and concentrating rehabilitation efforts in newer areas, where they believe it may be possible to reduce I/I at a lower unit cost.

Impact of Peak Flows on WWTP Operation

The Eugene-Springfield regional WWTP is relatively new, and was designed to incorporate considerable flexibility for handling flow variations due to wet weather and future growth. Design ADWF is 49 mgd; maximum design flow is approximately 180 mgd. At flows above 175 to 185 mgd, raw wastewater bypasses at the pump stations and WWTP would be activated. Current ADWF to the WWTP is 22 mgd. During a recent storm period, a peak flow of 143 mgd (95 mgd daily flow) was reached. The flow from Springfield alone, however, cannot be reliably isolated because of the location and type of flow metering devices that were installed in the interceptor system.

Effluent requirements for discharge to the Willamette River are 30/30 mg/l BOD and suspended solids in winter and 10/10 in summer. The WWTP is an activated sludge plant, which is run as a contact stabilization process in winter and modified plug flow in summer. During peak flow periods, effluent quality is maintained, first by putting on line additional primary and secondary clarifiers, and then by bypassing a portion of the primary effluent around the secondary treatment process. Bypassing is generally required for only a few hours to one- half day. The flow to the secondary process can be controlled by pre- selecting the flow level at which bypassing will start. The disinfected combined primary and secondary effluent generally does not exceed 20 mg/l sususpended solids.

In addition to the capital cost for excess capacity, the major cost associated with treating peak wet weather flows is the increased labor required for clean-up of the additional clarifier units which must be put into service during the peak flow periods but are no longer needed after the flows recede. Increased energy costs associated with peak flows are fairly minimal, since the bypassing of the secondary process means that significant increases in aeration are not required. Because there is a "trade-off" between maintaining effluent quality and reducing clean-up requirements, there is a certain amount of guess work involved in deciding when to put additional clarifiers on line during peak flow periods.

Other wet weather impacts at the WWTP include problems caused by large quantities of grit which are washed out of the sewer system during the first large storm of the season, sometimes causing damage to equipment and plugged lines (the grit chamber is located downstream of the comminutor). Also during wet weather periods, solids washout may occur, resulting in decreased gas production in the digesters. Foreign materials washed out of the sewer system (oils, grease, etc.) also may inhibit bacterial action.

MILWAUKEE METROPOLITAN SEWERAGE DISTRICT, WISCONSIN

The Milwaukee Metropolitan Sewerage District (MMSD) serves 28 communities in the southeastern portion of Wisconsin. The largest of the communities is the City of Milwaukee. MMSD operates a 294-mile interceptor system and two treatment plants; the collection systems are owned and operated by the individual communities. The total MMSD service area includes over 2,800 miles of sewer mains, of which approximately 20 percent are combined storm/sanitary sewers, mostly located within the City of Milwaukee.

In the late 1970's MMSD initiated the Milwaukee Water Pollution Abatement Program to address the problems caused by inadequacies in the wastewater collection, transport, and treatment systems. These problems included overflows and bypassess from the interceptor and collection systems, sewage back-ups into building basements, and discharges of inadequately treated wastewater to Lake Michigan. The Water Pollution Abatement Program included major projects to upgrade the interceptor system and treatment plants, projects to address problems in the combined sewer service area, as well as a comprehensive SSES for the separate sanitary sewer portion of the service area. The SSES was completed in 1981.

Although the Milwaukee SSES identified direct inflow as a significant portion of peak wet weather flows, and much of the subsequent rehabilitation effort was concentrated on removing those types of sources, the study did include documentation and extensive field investigation of sources which the study termed "indirect inflow." These sources included leakage through manhole frame/chimney defects, as well as sources on private property, primarily foundation drains. The estimates of source flow contributions developed for the SSES indicate that more than 50 percent of the maximum hour I/I flow is due to these types of RII sources.

System Description

The MMSD service area is relatively flat. In general, the area drains west to east and north to south toward the Milwaukee River and Lake Michigan. Soils are typically clay, with more sandy soils in the western portion of the service area. Many of the soils are of glacial origin, resulting in seams of more permeable material throughout the soil mantle. The groundwater level is typically about six feet below the surface, and increases to about three feet in the spring.

The Milwaukee area receives approximately 31 inches of precipitation (water equivalent) annually. Rainfall occurs throughout the year, although it is lowest in the coldest months of January and February when most precipitation occurs as snowfall. In early spring, conditions of rain, snowmelt, and high groundwater occur simultaneously, resulting in the highest I/I flows. Freezing temperatures in the winter result in frost heave damage to streets and manholes.

The original sewers in the separate sanitary sewer system were constructed in the 1920's, with more recent construction in the outlying communities. The average depth of sewer mains is 15 to 20 feet; service laterals are typically 6 to 10 feet deep. A considerable portion of the system is therefore below the groundwater table. In the older portions of the service area, individual buildings are served by both storm and sanitary laterals, which have commonly been constructed in the same trench.

RII Documentation

Flow monitoring was conducted at several hundred locations during the SSES. Infiltration was identified as the early morning flow rate, and "inflow" was calculated as the difference between total storm flow and non-rainfall flow (base flow plus infiltration). Both infiltration and inflow were projected to a maximum condition using adjustment factors based on historical data from 34 permanent monitoring sites in the system. These factors were determined for different areas of the system by relating the measured infiltration and peak hour inflow during the monitoring period at the various permanent monitoring sites to the infiltration and peak hour inflow for selected maximum historical infiltration and inflow events. For the total system, the ratio of design PWWF to ADWF is approximately 7.5 to 1. Of the projected total peak hour flow of 1,155 mgd, 878 mgd or 76 percent is "inflow," i.e., rainfall induced I/I.

Extensive field investigations were conducted as part of the SSES to identify specific sources of I/I and quantify the flow contribution from each source. Physical inspections were conducted for all manholes in the SSES area, and included lamping of the inlet and outlet sewers from the manhole. The manhole inspections identified vented covers, misaligned and unsealed frames, manholes subject to ponding, and manholes and sewers with evidence of infiltration (leaks, deposits, roots). Building inspections were done to identify I/I sources on private property, including downspouts, roof drains, area drains, foundation drains, and sump pumps. Inspections were attempted at all residential and small commercial buildings; approximately 60 percent of the 165,000 attempted inspections were completed.

Smoke testing was conducted for all of the SSES area. Dye flooding was conducted in approximately 35 percent of the area. All storm sewers and drainage ditches which paralleled or crossed over sanitary sewers or laterals were included. Dye flooding identified both direct and indirect storm/sewer connections. Street flooding was conducted for about 10 percent of the manholes in the system in order to identify and quantify I/I which enters manholes through frame/chimney defects. TV inspection was conducted for about 13 percent of the system on those sewers identified as inflow or infiltration sources through dye flooding (medium to heavy transfer) or sewer lamping. In addition, two pilot projects were developed for in-depth investigation of I/I sources from manholes and from private property (laterals and foundation drains).

The private property I/I study found that flows from foundation drains and defective laterals were responsive to rainfall, with the maximum flows occurring during rain events when the groundwater was high. Indirect flow transfer from foundation drains and storm sewers and ditches was identified as a significant source of I/I in defective laterals. TV inspection of the laterals (primarily pre-1960 VCP with mortar joints) indicated that two-thirds of the joints were defective. The direction of surface drainage and location of downspout discharges were other factors cited as influencing lateral and foundation drain flows.

The major sources of I/I identified through the SSES were manholes (97 percent with vented covers and 59 percent with misaligned frames) and foundation drains. Distinction was made between inflow and infiltration sources, and between direct and indirect inflow sources. Indirect inflow (RII) sources include manhole frame/chimney leakage and manhole, sewer, and lateral defects detected through rainfall simulation (smoke testing and dye flooding), including indirect flow transfer from storm sewers. Manhole frame/chimney leakage occurs when surface runoff seeps into cracks and joints in concrete streets and enters manholes with unsealed or misaligned frames. This phenomenon is caused by freezing and thawing, which create gaps between the frame and chimney and in the street pavement. In some manholes which have been excavated for repair, large voids or channels have been found around the manhole frame, created by water infiltrating to the defects.

Estimates of flow contribution were developed for each type of I/I source. For direct inflow sources, the rational method was used. For indirect inflow, flow estimates were based on flow rates measured during dye flooding and street flooding. The total maximum hour I/I calculated in this manner is approximately 800 mgd, of which 32 percent is attributable to direct inflow sources, 40 percent to foundation drains, 12 percent to manhole frame/chimney leakage, and 15 percent to infiltration through laterals, sewer mains, and manholes, a portion of which was identified through rainfall simulation and therefore can be considered to be RII. Approximately 60 percent of the peak I/I flow appears to be due to RII.

A site visit was made to Milwaukee during the course of this study, and District and community staff were interviewed regarding RII problems in the sewer system. Staff indicate that RII from laterals may have been underestimated in the SSES. Dye flooding work during the SSES identified considerable flow transfer from storm drains crossing over laterals. In particular, the common trench storm and sanitary laterals that are typical in the older portions of the service area are potential sources of indirect flow transfer. Some of these types of sources were demonstrated as weak smoke emissions from roof leaders that were presumably properly connected to a storm lateral, which was then exfiltrating to the sanitary lateral.

RII Control Program

The I/I correction work resulting from the SSES consisted primarily of eliminating direct inflow through manhole covers and indirect inflow (RII) through manhole frame/chimney interfaces. The program also included the correction of illegal clear water connections to the sanitary sewer from private property (other than foundation drains), some sewer main grouting, correction of connected catch basin leads, and bulkhead repairs. The District conducted a manhole rehabilitation pilot project to evaluate different methods of correcting manhole frame/chimney leakage. (A more detailed discussion of these manhole frame/chimney rehabilitation methods is presented in Appendix D.)

None of the service lateral rehabilitation that was recommended in the SSES was done due to legal ramifications. Two of the communities have sucessfully addressed foundation drain sources. The Villages of Brown Deer and Menomonee Falls have enforced the disconnection of the foundation drains identified in the SSES and have instituted ordinances requiring inspection and disconnection of illegal sanitary sewer connections (foundation drains, sump pumps, downspouts). The Brown Deer ordinance requires conformance before the property can be sold. The Menomonee Falls ordinance empowers the plumbing inspector to enter a property, upon identification, to ascertain the quantity, quality, and condition of sanitary and clear water discharges, and provides the authority to require disconnection within six months of written notification of violation of the ordinance. Brown Deer has taken further steps to identify illegal connections in buildings that were not inspected during the SSES.

All of the communities in the District have prevailed upon property owners to correct illegally connected sump pumps and area and roof drains, at least for those properties inspected during the SSES. All communities have also adopted ordinances that prohibit clear water connections to the sanitary sewer system. However, only the two communities identified above have gone beyond the SSES-recommended private property rehabilitation program to address illegal connections on properties not inspected during the SSES and foundation drain connections that existed prior to adoption of the ordinance.

The District is in the process of implementing a long-term control program to monitor I/I levels throughout the system and track the impact of rehabilitation work. Permanent monitors with telemetry are installed at approximately 50 locations, and further phases of the program will include 100 to 150 monitoring sites for smaller areas within the communities. The data collected from the long-term monitoring program will be used to identify specific areas which continue to have particularly high I/I flows so that correction work can be planned.

Impact of Peak Flows on WWTP Operation

The District operates two major wastewater treatment plants: Jones Island and South Shore. Both plants discharge to Lake Michigan. The Jones Island plant service area includes the combined sewer portion of the system, as well as portions of the separate sewer system.

The District's interceptor system includes 20 diversion structures (with 15 more planned), which are used to control the flow during wet weather periods. Nine of these diversion chambers are automatically controlled based on the monitored hydraulic grade line at key points in the system, as well as monitored precipitation. Because of the size of the District, precipitation and flow trends must be monitored closely and are used to trigger the activation of flow diversions. The interceptor diversions are used to divert flows from Jones Island to the South Shore WWTP during peak flow periods.

The Jones Island WWTP is currently undergoing expansion to increase maximum day capacity from 200 mgd to 330 mgd. The design maximum hourly flow is 390 mgd, of which 330 mgd receives full treatment and 60 mgd is in-line flow receiving only disinfection and dechlorination. Current average flows during low rainfall months are in the range of 115 to 135 mgd. Prior to the current expansion, the plant was limited by secondary clarifier capacity. Flow was taken until secondary clarifier blankets were in danger of spilling into the effluent. When high secondary clarifier blankets were observed, a portion of the flow was bypassed to prevent solids carry-over into the effluent. All bypassed flows received primary treatment and were chlorinated and dechlorinated before discharge.

The current operating strategy for high wet weather flows includes diverting flow to the South Shore WWTP and controlling the influent flow through two (low- and high-level) siphon gates at the entrance to the plant. Throttling of these gates backs up the flow into the collection system. The objective of system operation during high flows is to maintain the sludge blanket in the secondary clarifiers and avoid spilling solids into the effluent. Standard procedures and criteria for wet weather operation have been developed and are followed during periods of high wastewater flows. After the plant expansion is completed, return sludge capacity rather than clarifier capacity could become the limiting factor in handling high flows during bulking sludge conditions.

The South Shore WWTP has an average flow of 80 to 90 mgd, but may experience peak flows in excess of 450 mgd. A typical "good-sized" storm will produce flows of 300 to 350 mgd. There is a significant lag time in the sewer system, with normal dry weather peak flows reaching the plant eight hours after the time of peak system flow.

During wet weather, the flow through the plant is increased to avoid back-ups in the collection system. This is done by increasing the grit channel velocity by opening the butterfly valves. Primary clarifiers that may have been out of service for maintenance or repair are put back on line. Flow through the plant is limited by secondary clarifier capacity, which is normally 240 mgd. (Higher flows, up to 260 to 280 mgd, may be put through the secondaries, depending on the level of the lake.) Primary effluent in excess of this amount is bypassed around the secondary plant, and the combined effluent is chlorinated. Although the bar screens can be bypassed, all flow must go through the primary basins. Bypassing of the secondary plant often occurs for several days during prolonged wet weather periods. A new plant expansion currently under construction will add eight final clarifiers, thereby increasing capacity through the secondary process by 50 percent.

NORTHEAST OHIO REGIONAL SEWER DISTRICT, OHIO

The Northeast Ohio Regional Sewer District (NEORSD) includes 41 communities in the Cleveland, Ohio, metropolitan area. NEORSD is divided into two major subdistricts: The City of Cleveland, which has a combined sewer system; and the surrounding communities, which have primarily separate systems. NEORSD operates an interceptor system and five treatment plants; the collection systems are owned and operated by the individual communities. Most of the separated portion of the system is contained within two major planning areas, the Easterly Separate Sewer Area (ESSA) and the Southwest Interceptor Area (SWIA), for which SSES's were completed in 1983 and 1984. The oldest separate sewers in the District are located in the ESSA. Together, the ESSA and SWIA contain approximately 1,200 miles of sanitary sewers serving a population of about 500,000.

Overflows and bypasses occur at over 200 locations in the separate sewer system, most activated by rain events of less than 0.2 inches per hour. Regulator chambers in the interceptor system are used to restrict the flow to the WWTP's. Basement back-ups are a major problem during wet weather.

System Description

The topography of the area ranges from flat to fairly steep, the higher elevations located on a glaciated plateau in the northeastern portion of the District in the ESSA. Numerous streams drain the service area, with drainage generally toward the Cuyahoga River and Lake Erie. Soils consist primarily of moraine deposits of clayey silt and gravel. The predominant soil association is characterized by very slow permeability (less than 0.2 in/hr) and a seasonally high groundwater table from November through June. The groundwater level is typically six to ten feet below the surface;

however, high bedrock exists in some areas, with perched groundwater at 12 to 30 inches below the surface. Rainfall occurs year round and averages 35 to 40 inches, depending on location. Both localized and area-wide storms can occur in the system.

Construction of the original separate sanitary systems began around 1915, with a majority of the sewers constructed during the first part of the century. Most of the sewers are clay, with mortar or bituminous joints in the older pipes and compression-type joints used since 1965. Most older manholes are brick, with concrete manholes being constructed since 1970.

Service laterals are also predominantly clay pipe, and are typically constructed in the same trench as storm laterals. Almost all buildings in the service area have storm laterals to convey roof and foundation drainage to the storm sewer system. Direct foundation drain connections to the sanitary system are not common, since storm laterals are generally deep enough to collect foundation drainage without the need for sump pumps.

The oldest sanitary sewers, constructed prior to about 1930, were installed in common trenches with storm sewers. Over 50 percent of the separate system (80 percent of the ESSA) consists of common trench sewers. There are two basic types of common trench construction: dual system (side-by-side) and over-under. In the dual system, the storm sewer was typically laid next to and about one foot higher than the sanitary sewer. This was generally done by digging a single wide trench and refilling the bottom of the trench on one side to form a bench for the storm sewer. The entire trench was filled with granular backfill; porous slag material was often used as bedding and fill material between the storm and sanitary sewers. The two sewers were generally accessed by separate manholes; where common manholes existed, they were separated by either partial or full-height walls. However, the sewers are so close together that the storm pipe walls are usually visible in the sanitary sewer manholes.

In the over-under sewers, the storm drain is laid on top of the sanitary sewer, often with less than one foot clearance between the top of the sanitary pipe and the bottom of the storm sewer. In many cases the fill material between the two pipes has eroded, which causes settlement of the storm sewer and structural damage (springline cracks and potential crushing) to the underlying sanitary pipe. The over-under sewer manholes were generally constructed with a steel or cast iron plate separating the access chambers of the storm and sanitary pipes. These plates easily leaked water between the two sewers and could be dislodged during high flow conditions. Some of these plates have been grouted or welded to alleviate this problem.

RII Documentation

Flow monitoring was conducted at 85 long-term monitoring sites and over 1,100 short-term sites. Flow data were analyzed to determine base infiltration and peak (rain-induced) I/I. Rain induced I/I flows were projected to a design storm condition based on rainfall intensity. These flow projections resulted in ratios of ADWF to PWWF of over 20 to 1 in the ESSA and approximately 12 to 1 in the SWIA. Over 90 percent of the PWWF is due to rain induced I/I.

Field investigations conducted during the SSES's included smoke testing, dye flooding, and TV inspection. Smoke testing was conducted in approximately 30 percent of the system. In many cases, the joints in the common trench storm and sanitary sewers were found to leak so badly that the smoke could not reach inflow connections, such as drains on private property. Also, it was often difficult to distinguish between direct and indirect connections on private property (e.g., roof downspouts) because of leakage between common trench storm and sanitary sewer laterals. Dye flooding indicated that the flow transfer between the storm and sanitary systems was rapid. In over-under systems, the peak flow in the sanitary sewer was reached within 10 minutes; in side-by-side and separate trench sewers, within 20 to 30 minutes. Lamping and TV inspection of the sewers indicated that most of the leaks were from joints and service connections.

Estimates of direct inflow, based on smoke testing and dye flooding, could account for 5 to 15 percent of the peak wet weather flow. It was concluded that the remaining rainfall induced flow was due to rapid infiltration, primarily due to exfiltration from leaky storm drains and storm laterals into sanitary sewers and laterals.

A site visit was made to NEORSD during the course of this study, and District staff were interviewed regarding RII problems in the sewer system. Staff identified potential RII sources in service laterals, particularly the connection to the main, including hammer tap connections; the steeper grade and often vertical drop of the lateral to the main connection; traffic loads; and the greater hydraulic pressure on the lower portion of the laterals when the sewer main is surcharged.

RII Control Program

The primary emphasis of the District's program is construction of new interceptor sewers. Each community within the District is responsible for its own rehabilitation program. With few exceptions, the rehabilitation work is addressing only the public portion of the sewer system. Since cross leakage between storm and sanitary sewers, particularly with common trench construction, is the major source of I/I, correction efforts are concentrated on rehabilitation and flow regulation in the storm sewer system, as well as sanitary sewer rehabilitation. Work includes separation of common trench sewers by construction of new storm sewers, addition of storm sewer capacity, and rehabilitation of common trench storm/sanitary sewer manholes (constructing walls in manholes between side-by-side sewers, sealing plates in over-under sewers). Vortex regulators are being used in many communities to restrict the flow into the storm drain system. The impetus for these types of solutions is to eliminate basement flooding. Essentially, the vortex regulators restrict storm flows from entering the storm sewers, causing temporary flooding on the streets. This reduces the load on the storm drain system and thus reduces overflows and indirect flow transfer to the sanitary system.

The District is coordinating several pilot rehabilitation projects in various communities. Each community is responsible for the rehabilitation work, and pre- and post-rehabilitation flow monitoring is conducted by the District. Each of the pilot areas includes approximately 2,500 feet of pipe and 100 laterals. The evaluation of the results of the pilot projects has not yet been completed.

Impact of Peak Flows on WWTP Operation

The District operates three major wastewater treatment plants, called Easterly, Westerly, and Southerly, based on their respective locations within the District service area. The Westerly and Easterly plants discharge to Lake Erie. The Southerly plant discharges to the Cuyahoga River. All of the plants receive some amount of combined sewer discharges. The majority of the combined sewer flows go to the Westerly plant, which includes a CSO treatment facility. The Easterly plant, which serves a portion of the combined sewer area, as well as the ESSA with a large proportion of common trench construction, has a wet weather capacity of 330 mgd. Flows in excess of that amount are bypassed to Lake Erie.

The Southerly plant has undergone a recent expansion to provide up to 400 mgd of two-stage secondary capacity (plus filtration), with an additional 335 mgd of primary only treatment capacity for peak wet weather flows. During the period prior to the new expansion coming on line, it was found that peak flows could be handled by using some of the considerable primary tank capacity for equalization. Problems with solids washout in the final filters can occur under high flows, therefore requiring flow blending to reduce the solids concentration in the final effluent.

CITY OF BATON ROUGE, LOUISIANA

The City of Baton Rouge, Parish of East Baton Rouge (Baton Rouge) is located in the southeast portion of Louisiana along the Mississippi River. The Baton Rouge system serves a population of about 450,000 and includes approximately 1,500 miles of sewer main. The sewer system is divided into four major areas, three of which comprise the original Consolidated Sewer District (CSD) and the fourth area being the suburban area, located south, east, and north of the original CSD. The original CSD region is further divided into the North, Central, and South Districts. Each region in the CSD has its own treatment plant that discharges to the Mississippi River. The suburban area has 144 local wastewater treatment facilities that discharge to local streams that flow to either the Amite River or Mississippi River.

In the late 1970's an extensive SSES program was conducted in the area served by the three main treatment plants. The results of that study indicated that I/I is "excessive" in the collection system. Overflows and bypasses occur throughout the collection system during high intensity storm events that are common in the area. The SSES indicated a large number of direct connections between the sanitary and storm sewers at that time. Errors found in the original SSES work led to verification field work performed in 1987-88 in four pilot areas in the collection system. The results of the pilot program indicate that the majority of I/I sources are defects in the sewers, not direct connections. Only sixteen potential direct connections have been found in the pilot areas during the additional field work. The City staff believe that a large portion of the I/I is coming from the shallow private laterals.

System Description

Most of the service area is relatively flat, averaging 45 feet above sea level. Most of the surface drainage in the area flows east into the Comit or Amite Rivers, where as most of the sewage flows to the west for treatment and discharge into the Mississippi River. As most of the City is located in the flood plain between two rivers, the soils are primarily alluvium (Mississippi deposit), which are fairly permeable. The remaining soil is nonexpansive clay, with a low permeability. Bedrock is several thousand feet below the ground surface. The groundwater is usually below the sewers except in those areas immediately adjacent to the Mississippi, Amite, and Comite Rivers.

The climate is typical of states around the Gulf of Mexico, with about 54 inches of rainfall per year occurring throughout the year. Average monthly rainfall is 3 to 5 inches, and storm events range from high intensity short duration thunderstorms to more protracted rainfall from hurricanes and other tropical storms. Peak intensities of greater than one inch per hour are fairly common for storms in the area.

The sewer system was originally constructed in 1890 with clay pipe. Sewers constructed up to 1960 were constructed with clay pipe and cement mortar or asphalt poured joints. Beginning around 1960 concrete pipes were installed for a major portion of the collection system for all pipe sizes including service laterals. Approximately 80 percent of the new sewers constructed in recent years have been installed in backyard and cross country easements and drainage corridors.

Joint construction in the 1960's shifted to rubber gaskets. In recent years, PVC has been used extensively in smaller sewers because of its ease of installation. Creek crossings and canal crossings are made with cast or ductile iron pipe. Sewer mains are typically placed on the opposite side of the street from the storm sewer with pipe crossings at intersections and catch basins. The depth of the sewers ranges from 4 to 20 feet, at which point a pump station is normally constructed. Service laterals range in depth from the ground surface to about three feet at the curb. Service lateral are constructed with a six-inch pipe from the main to the curb line and a four- or six-inch upper lateral from the curb to the building. There are approximately 105,000 service connections in the system, for an average lateral density of 13 per 1,000 feet of main.

RII Documentation

During the earlier SSES work flow monitors were placed at key pump stations and bypasses throughout the CSD area. The flow monitoring and subsequent field work indicated what was believed to be inflow resulting from direct cross connections to storm sewers, drainage crossings, and manhole leakage. The PWWF (hour) to ADWF (day) ratio ranged from 4 to 8 depending on the District. The peak flows were projected to a 6-inch, 24-hour duration storm.

Follow-up investigations of the early SSES work showed many inconsistencies between the data and the results presented, so new field work was performed in four pilot areas in the CSD service area, containing 93,000 feet of sewers. The majority of the field work conducted in the pilot areas was smoke testing since it proved to be most effective during the earlier SSES work for finding defects in the sewers. The results of the smoke testing in particular showed a dramatic increase in the number of smoke testing returns than had been detected during the earlier SSES work, and approximately 16 potential cross connections. The estimated potential peak day I/I flow from each pilot area was 2.3 to 2.8 times the estimate of the earlier SSES work.

More than 600 defects were located during the pilot program compared to 157 in th earlier SSES for the same areas tested. The greater than two times increase in the I/I flow estimates for the pilot areas may be attributable to the differences in field procedures and that the 1988 field work was conducted during drought conditions. Drought conditions provided the maximum dryness of the soil, allowing more smoke to reach the surface from defects in the pipes.

Each smoke return was classified by type and location of defect, and amount of smoke observed. Based on the three observations, the defect was then assigned an estimated I/I flow value that was used to calculate the estimated peak flow for each pilot area. The data was summarized to show the percent of leaks detected and percent of I/I contributed by main line, service line (laterals), and manhole leaks. For three of the four pilot areas, the estimated I/I flows from the sewer mains and laterals were 85 percent or greater and the fourth area had 63 percent from the mains and laterals. The estimated I/I from the laterals ranged from 9 percent to 58 percent with an average value of 32 percent. Based on the information from the pilot areas, the main lines contribute the majority of the I/I to the collection systems, with the laterals also contributing a significant portion of the I/I. The majority of the defects found during the pilot program appear to be from RII with only 16 defects suspected of being direct connections. The earlier SSES work apparently included both direct connections and indirect flow transfers in the inflow estimates.

Television and manhole inspection of the sewers during the earlier SSES work concluded that the mains were generally in good structural condition except at the joints. The pipe joints in many cases were offset or open, and lateral connections to the mains were often cracked, protruding, or otherwise improperly sealed. To date no television inspections have been performed on the laterals to determine structural condition.

A site visit was made to Baton Rouge during the course of this study, and City staff and consultants were interviewed regarding RII problems in the sewer system. With respect to the condition of the sewers, those present identified mains and laterals as being the primary sources of RII in the collection system. Specific problems are connections between the lateral and the main, connections at manholes, and location of sewer lines either in easements or alongside drainage ditches. Roots are a common problem in the Baton Rouge area, particularly in easement areas. The roots expand the size of a defect once the root has made an entrance into the sewer. Based on the types and locations of smoke returns, it would appear that soil channels to the sewer defects may be the primary RII pathway. The French drain effect of the backfill in the trench was felt to be of minor significance except in local areas where the predominant soils are clay.

The smoke testing in the pilot areas found many cave-ins above and next to sewer mains ranging in size from 6-inches to over 24-inches. Defects of this size adjacent to drainage ditches or along curbs and gutters allow large amounts of RII into the collection system. The City currently has no routine maintenance program other than responding to emergency problems. The current backlog of over 600 defects and cave-ins means that only the worst defects can be addressed. City staff felt that a good maintenance program would greatly aid the reduction of RII in the system.

RII Control Program

As a result of the early SSES and subsequent cost effectiveness analysis, limited rehabilitation work was performed but no reduction in I/I flows were noted. The current pilot program is currently in the design phase to rehabilitate all the main line defects identified during the field testing program. At this time, City staff projects that a 40 percent reduction in I/I will be achieved using this type of rehabilitation approach. The City is also looking into expanding the current rehabilitation program to include work on the service laterals.

Rehabilitation techniques used in the past in the CSD area have consisted of most of the currently available techniques including, slip-lining, inversion lining and pipe replacement. Rehabilitation techniques being considered for the pilot program include point repairs, pipe replacement, slip-lining, and manhole sealing. Results from the pilot program are anticipated to be available within a year.

The City has re-assessed its approach to I/I and feels that a long-term solution is required to properly achieve long-lasting results. The early SSES work performed by the City was conducted in a compressed time frame and the results could not be verified or repeated. As part of the City's overall plan, all wastewater from the suburban areas will be treated and disposed of at either the

North or South wastewater treatment plants to eliminate discharges to the Amite River Basin. Also one of the goals of the I/I reduction program is the elimination of at least 40 known bypass locations.

Impact of Peak Flow on WWTP Operations

The city operates three main wastewater treatment plants and has another 144 smaller wastewater treatment facilities within its jurisdiction. The three main plants discharge to the Mississippi River and the smaller plants discharge to stream and sloughs in the Amite River Basin. The three main WWTP's are the North CSD, Central CSD, and South CSD plants; the smaller wastewater treatment facilities are referred to as the "suburban plants". A brief discussion of the operation of the three main plants follows.

North CSD WWTP. The North CSD plant was recently rehabilitated and upgraded from primary to secondary treatment with a design process capacity of 8 mgd and a hydraulic capacity of 23 mgd. To date all wet weather flows that reach the plant can be treated. The plant has been designed to allow bypassing of peak flows in excess of the 23 mgd peak capacity. Some bypasses exist upstream in the collection system, but the need to bypass plant flows has never occurred. The projected design (year 2010) peak hour wet weather flow to the plant is about 47 mgd.

The final effluent limits for the North CSD plant are 30 mg/l BOD and 30 mg/l TSS. The plant has just come on line recently after being upgraded from primary treatment only. The new process at this plant uses trickling filters to achieve secondary standards. The current ADWF for the plant is about 6 mgd. The total PWWF to ADWF ratio for this plant is projected to be 7.

The major cost associated with treating the wet weather flows is increased labor required to operate the plant under peak flow conditions. Power costs do not increase significantly since the final discharge is a gravity outfall. Chlorination use is also increased and therefore is more expensive than during dry weather flow operations.

Central CSD WWTP. The Central CSD plant was constructed in 1960 and upgraded to secondary treatment in 1978. The secondary portion of the plant has a process capacity of 20 mgd, and the overall hydraulic capacity of the primary section of the plant is 40 mgd. Current operation of the plant during wet weather is to process between 20 and 23 mgd through the secondary system with the remainder of the flow receiving only primary treatment. The influent flow meter to the plant peaks at 40 mgd, but City staff are certain that higher flows have come through the system. Peak projected flows for the Central region are approximately 55 mgd, and bypasses do occur in the collection system. Current ADWF at the plant is 15 mgd. The ratio of PWWF to ADWF for the plant is approximately 2.7.

Discharge parameters for the plant are the same as those for the North CSD plant. Even during wet weather the seven day averages for both BOD and TSS have been met without difficulty. The basic secondary process flow train is a high purity oxygen activated sludge system with secondary clarification. Secondary effluent BOD during peak flow is sometimes high due to solids loss over the weirs at the clarifiers.

The costs associated with operating the treatment plant during wet weather consists of increased labor and power costs. Final effluent is pumped to the Mississippi River for discharge. Higher flows increase chemical costs particularly for oxygen and chlorine.

South CSD WWTP. The South CSD treatment plant was constructed in 1962 as a primary plant and is currently being upgraded to a secondary process. The secondary process will consist of trickling filters to bring the final effluent into compliance with discharge requirements. Discharge requirements are the same as those for the North CSD plant. The ADWF for the plant is currently about 14.5 mgd; the PWWF to ADWF ratio is approximately 3.5. The current capacity of the plant is 16 mgd, and the plant can handle up to 30 mgd peak flows. The 30 mgd peak flow limit is caused by the limitations of the effluent pumps.

Three major bypasses exist upstream of the treatment plant, so true peak flows in the collection system never reach the plant. Other than the effluent pumps, the hydraulic capacity is estimated at greater than 50 mgd. When the suburban area connects to the CSD system the majority of the flow that went to the many small plants will go to the South CSD plant. This connection is scheduled to take place by 1994.

The costs associated with operating the treatment plant during wet weather flows are labor and power with some additional cost for chlorine. With the secondary treatment plant on line the costs should not increase significantly for wet weather flows, since the plant will have trickling filters for the secondary process.

CITY OF SPRINGFIELD, MISSOURI

The City of Springfield is located in southwestern Missouri. The wastewater service area is divided into two main drainage basins, each served by a separate WWTP. The larger of the two basins is the Southwest area, which includes approximately 80 percent of the City, including the oldest portions. The Southwest system includes over 500 miles of sanitary sewers serving approximately 160,000 people.

Problems due to I/I include surcharging and overflows in the collection system and basement flooding. Overflows occur at approximately ten sites during any good-sized storm, and at 100 or more locations during large rainfall events. During the period 1957 through 1973, the City conducted various studies and programs to address I/I in the sewer system. For the most part, these efforts addressed only the most obvious of the I/I sources identified, such as defective manholes and sewer mains with major problems. In 1974, the City completed an I/I Analysis in conjunction with its grant project to expand the Southwest WWTP. The I/I Analysis determined that the ratio of PWWF to ADWF was approximately 6 to 1 as measured at the WWTP, and as high as 8 to 1 if the estimated overflows in the system were included. The peak flow reaching the WWTP during large "general" storms falling under saturated soil conditions was found to be about twice that for isolated storms falling under relatively dry soil conditions. It was concluded that half of the initial peak I/I flow was due to infiltration and "indirect inflow" (foundation drains), and that this peak flow was sustained for several hours after rainfall, with virtually all of the sustained peak due to RII. Although subsequent smoke testing revealed some direct connections from roof and yard drains, which were corrected, the system continues to experience high peak flows during wet weather.

System Description

The service area is characterized by relatively flat to gently rolling terrain located on the Springfield Plateau, which forms the western flanks of the Ozark Mountains. A bedrock of limestone lies beneath this plateau at an average depth of six to eight feet. Above the bedrock is a four- to six-foot band of clay soil that has developed from the weathering of limestone. In about half of the area, an impermeable layer of soil, called the fragipan, is located above the clay band at a depth of about two to three feet below the surface. The topsoil consists of silty loams. The solubility of the limestone creates crevices, caverns, and springs in the underlying bedrock and results in sink holes on the surface in many locations.

Although the perennial groundwater table is at least 25 feet deep, about 30 percent of the area is characterized by a perched water table which rests atop the bedrock or impermeable fragipan. Sink holes and crevices in the limestone create underground passageways for water. Average annual rainfall is approximately 41 inches, with May and June being the peak rainfall months. The area experiences both localized thunderstorm-type events, as well as more general, longer duration storms.

The original sewer system was constructed during the period 1894 to 1911. Roughly half of the sewer system is constructed of older type VCP with mortar joints and brick manholes. The remaining half of the system has been constructed over the past 30 years with newer, improved joint materials and precast concrete manholes. Service laterals are generally of similar construction as the mains. The sewer mains are typically six to eight feet deep; therefore, a substantial portion of the sewer trenches extend into the bedrock. Only a small portion of the City is served by a storm sewer system. Surface drainage is generally carried by overland flow along street gutters, ditches, and natural drainage channels. Roughly 20 percent of the buildings in the City have basements and foundation drains.

RII Documentation

I/I flows within the sewer system were documented through flow monitoring during the SSES, which was completed by City staff in 1980. Ten areas of the system were selected for monitoring, based on known I/I problems. For the measured storms, maximum daily wet weather to average dry weather flow ratios for the individual monitored subareas ranged from about 5 to 20.

The field investigations conducted during the SSES included smoke testing, dye flooding, and manhole inspection. The smoke testing and dye flooding identified relatively few sources, primarily because direct inflow connections (roof and yard drains) had been identified and corrected under previous programs. Some indirect connections between storm and sanitary sewers were located and corrected, and some smoke returns were observed from sewer mains. It was generally felt that the soil may not have been sufficiently dry to detect pipe defects in mains and laterals. The manhole inspection work primarily identified sources of infiltration through manhole walls and inverts. Television inspection conducted since 1966 throughout the system with the City's own equipment identified lateral taps and laterals with clear water discharges (from lateral defects or foundation drain connections) as specific sources of infiltration.

Although specific flow estimates based on source detection work were not developed as part of the SSES, the I/I Analysis did attempt to quantify the total flow contributions from direct inflow versus infiltration sources. Estimates of direct inflow were developed based on the observation of rain-induced I/I flows for isolated thunderstorm-type events during relatively dry soil conditions. In these types of storms, it was observed that a relatively high peak flow was reached in direct response to rainfall, and the peak receded quickly after the rainfall stopped. It was assumed that this response was due primarily to direct inflow sources, and a relationship between inflow volume and rainfall volume was developed. This relationship was then used to quantify the direct inflow portion of the flow for a large, general-type rainstorm falling under saturated soil conditions. It was found that direct inflow could account for approximately 40 mgd of the 84-mgd peak flow. However, the flow was sustained at nearly its peak level by infiltration alone for several hours after the rainfall had stopped and direct inflow subsided. This sustained peak could not be accounted for as storage in the system. Rather, it was theorized that during the initial peak storm period, the amount of RII entering the system was physically limited by the capacity of the system, but reached its peak flow rate only after the direct inflow had subsided.

The 1974 I/I Analysis also analyzed the flow trends in the system during the previous 13 years. It was found that both I/I as a percentage of total flow and as a percentage of rainfall had increased steadily over that period. The increased severity of I/I was attributed to both deterioration of the existing sewer system, as well as inadequate quality of construction of new sewers. On an annual basis, I/I was calculated to be about 15 percent of total precipitation and approximately 25 to 30 percent of effective precipitation (total precipitation minus evaporation).

Factor affecting RII in Springfield may include inadequate storm drainage and the hydrogeologic characteristics of the area. Because of the perched water table which exists in many portions of the system and the sinkholes and crevices characteristic of the underlying limestone bedrock, storm water can easily and rapidly be transmitted to sewer and lateral trenches and foundation drains. Although most buildings do not have foundation drains, a single foundation drain or defective lateral could be draining the water for a much larger area.

RII Control Program

The City has conducted sewer grouting since 1972, particularly in older areas of the system, with little success in reducing peak wet weather flows. As part of the SSES, a pilot rehabilitation project was conducted in a newer area of the City (sewers constructed after 1968) which experienced very high flows during rainfall periods. During a three-day heavy storm period prior to rehabilitation, the total flow volume was nearly seven times the normal dry weather flow. Manholes in the rehabilitation area were inspected during this storm period, and those that exhibited significant infiltration were subsequently sealed. The sewer main joints were air tested and grouted if necessary; however, most of the joints were found to be tight. During TV inspection, it was noted that many laterals were discharging clear water flows; however, no lateral rehabilitation was conducted as part of the project. The results of the pilot project indicated that although the infiltration through the rehabilitated manholes had been reduced or eliminated, the rehabilitation efforts had had negligible effect on the flows from the overall area.

Since the SSES, the City has allocated approximately 10 percent of its annual sewer budget for rehabilitation work, primarily slip-lining of isolated problem sewer reaches. Ongoing TV inspection is used to prioritize areas for rehabilitation. City staff believe that grouting has been ineffective in reducing RII, primarily because of migration of the RII to other sewer defects and to laterals. In general, they feel that sewer system rehabilitation is not cost- effective on a large scale basis.

Impact of Peak Flows on WWTP Operation

The existing Springfield Southwest WWTP is an advanced secondary treatment facility with nitrification, effluent filtration, and ozone disinfection with discharge to Wilson Creek. Effluent discharge limits are 10/10 mg/l BOD and suspended solids and 2 mg/l ammonia. The plant utilizes equalization basins during peak flow periods. Under high flow conditions, however, the plant sometimes experiences problems meeting the suspended solids and ammonia discharge limits. Currently, the equalization basins have limited capacity during extreme flow events. The State of Missouri is considering amending the City's discharge requirements to allow discharge from the equalization basins after some settling, to be dependent on stream flow and stream water quality.

NORTH AND SOUTH SHENANGO JOINT MUNICIPAL AUTHORITY, PENNSYLVANIA

The North and South Shenango Joint Municipal Authority includes the Townships of North and South Shenango, located along the shoreline of Pymatuning Reservoir in northwestern Pennsylvania. The Authority operates a collection system and treatment plant which serve a permanent population of about 1,200 and a summer population of approximately 6,000. The collection system includes approximately 90 miles of sewer mains and several pump stations.

The wastewater system was originally constructed in 1978. The WWTP and pump stations were designed based on a per capita unit flow of 100 gpd with a peaking factor of 2.5. Despite the fact that the contract specifications for the collection system pipelines specified a maximum allowable infiltration rate of 75 gpdim, the wet weather flows in the system have far exceeded design capacity, resulting in overflows at the pump stations and hydraulic overload of the WWTP. The current ratio of PWWF to ADWF is about 7 to 1.

The major wet weather problems occur in four areas in which the sewers were constructed of VCP with A-ring compression joints. The high I/I flows in the four problem areas are believed to be due to problems with the pipe joints, which appear to have been improperly manufactured and insufficiently tested. Apparently the joints lack "memory" and are forced out of position directly under external hydraulic pressure, although they may appear to be tight under internal air pressure testing.

During the course of the litigation against the original contractor and pipe supplier in the four problem areas, extensive field work was conducted in the system to document the infiltration problems. The results of this work indicated that rapid changes in water level in the sewer trenches in response to rainfall were the cause of the high rate of infiltration in the system.

System Description

The service area lies along a lake shoreline and is therefore characterized by relatively high groundwater levels. At some points in the western portion of the area, nearer to the lake, the piezometric groundwater elevation may be at or above the ground surface. Groundwater levels are highest during the early spring. Soils vary throughout the area, ranging from fine silt and clay to sand.

As noted above, the sewer system is only 10 years old. The system was constructed under several separate pipeline contracts. The four contract areas which appear to be contributing the most I/I to the system were constructed of VCP. Two other areas were constructed under different construction contracts, one with VCP made by another pipe manufacturer and the other with PVC pipe. The lower portion of the service laterals (portion within the public right-of-way) are typically six-inch and constructed of the same material as the sewer mains. The upper laterals are typically four-inch PVC.

Because of the high groundwater and the fact that North and South Shenango are largely resort communities, few of the houses have basements. Therefore, many of the laterals are as shallow as three to four feet below the surface. Storm drainage in the communities is by ditch system. Many of the sewer mains are located directly under ditches or gutters along the side of the roadways.

RII Documentation

Field work in the collection system was included in a Sewerage System Evaluation conducted in connection with the litigation over the pipeline construction. The field work included flow monitoring, flow isolation, groundwater monitoring, and limited smoke testing. The major focus of the field work was to isolate and quantify the infiltration in the different pipeline contract areas, and to determine the relationships between groundwater level, infiltration, and precipitation.

Groundwater monitors were installed in sewer trenches at 144 locations in the collection system. These monitors were designed to measure the hydrostatic head over the pipe in the trench. In addition, shallow wells were drilled at four locations adjacent to sewer trenches to document the differences in water level between the trench and the undisturbed soil around the trench.

The groundwater monitoring information was used to develop maps of groundwater elevation contours at different points in time and to identify areas where the sewer system was submerged. In general, the groundwater levels were highest in early spring and decreased during the summer. A considerable portion of the sewer system was found to be submerged during the spring and early summer, particularly in the western portion of the service area near the lake. Comparison of the groundwater data in the eastern and western portions of the system indicated that the east-to-west sewer trenches appear to drain the groundwater from the undisturbed natural soil in the eastern portion of the area and transport it in the trenches toward the western portion of the area via a French drain effect.

At several of the groundwater monitoring sites, continuous recorders were used to monitor the response of groundwater level to precipitation. Data from the recorders showed that water level in the sewer trench can increase rapidly in response to rainfall. Increases of three feet (the limit of monitoring) within a few hours of the onset of rainfall were recorded at sites throughout the system. Water levels seemed to be the most responsive to rainfall during the winter and early spring, and also responded to the daily thawing and snowmelt which occurred during parts of the winter. Sewage flows, as measured by flow monitors in the system, also correlated with increases in groundwater level in response to rainfall. Sewage flows increased during rainfall and were sustained for some time after the end of rainfall, indicating increased infiltration resulting from the precipitation.

In another study, measurements were made of infiltration from individual sewer pipe joints at six locations in the system. This procedure involved isolating the joints with a packer assembly and quantifying the infiltration rate under a range of piezometric heads (measured as the differential between the piezometric pressure in the trench and the inside of the sewer pipe). The infiltration rate clearly increased as the head differential increased, with rates ranging from 10 to as high as 2,600 gpd. The magnitude of the infiltration response varied from location to location.

It was noted that higher infiltration rates were observed in sewers installed in trenches underneath ditch lines. Many of these ditches have been observed to be flowing with water over one foot deep during storms, which can easily percolate into the disturbed soil and permeable backfill material in the underlying trench. It was also noted that laterals and lateral connections did not appear to be contributing significant extraneous flows, based on TV inspection conducted in conjunction with subsequent rehabilitation work.

Because the sewer system was constructed so recently, it is unlikely that any significant direct inflow sources exist in the system. This was confirmed by limited smoke testing that was conducted for the study, in which only one potential surface water inflow source was detected. Therefore, it can be concluded that the high wet weather flows in the sewer system are due to infiltration, and the flow increases during rainfall are due primarily to the increase in infiltration into defective sewer joints as a result of the increased groundwater level in the sewer trenches.

RII Control Program

As part of the work to evaluate methods to solve the infiltration problems in the sewer system, some sewer grouting was conducted in the system. The grout appeared to seal the joints internally, but infiltration was not reduced. It is believed that the inherent problems with the joint compression rings made grouting ineffective in preventing leakage due to external hydraulic pressure.

A pilot slip-lining project was performed on 1,400 feet of sewer in the system, with flow monitoring before and after the rehabilitation work. The results of the project indicated that infiltration in the slip-lined sewer was completely eliminated through the rehabilitation work. As a result of the settlement of the construction litigation, the Authority is now slip-lining all of the mains and slip-lining or replacing the lower laterals in the four problem contract areas. Due to the lack of rain this year, evaluation of the flow reduction resulting from the rehabilitation work has not been completed, although early results indicated substantial reductions in those areas rehabilitated in March 1988, prior to the drought period.

Impact of Peak Flows on WWTP Operation

The North and South Shenango WWTP is an activated sludge plant, which was designed for an average flow of 1.2 mgd and a maximum flow rate of 3.0 mgd. The plant consists of three separate 400,000-gpd contact stabilization units. It was envisioned that only one unit would be operated in the wintertime, and the additional units would be put into operation to handle the increased summertime population. Although the current wintertime service area population is less than 15 percent of the design maximum population, the high flows in the system to date have forced the operation of all three process units, even during the winter months. Since overflows and bypasses occur in the system during peak flow conditions, the current total peak wet weather flows cannot be measured, and the entire flow does not reach the WWTP.

During high flow periods, the influent is so dilute that it often meets the discharge limits for the plant effluent. The major problem, aside from lack of available capacity for future growth, is that the plant cannot meet the NPDES permit treatment performance requirements for 85 percent removal of BOD, due to the extremely dilute influent. The plant has also been flooded out a few times due to the high flows.

CITY OF AMES, IOWA

The City of Ames, Iowa, is located in central Iowa along the Skunk River. The collection system and treatment plant serve a population of approximately 45,000, including the Iowa State University campus, which comprises almost half of the total population. The collection system contains approximately 135 miles of sewers.

The City conducted an I/I Analysis and SSES during the late 1970's in conjunction with a facilities plan for expansion of the WWTP. During wet weather periods, the plant cannot handle the peak flows in the system, and the influent sluice gate must be throttled to limit the flow entering the plant, often for as long as several days. Several times each year during extremely wet conditions, bypassing of raw wastewater occurs both at the plant and at several points in the collection system. Basement backups during wet weather also occur as a result of high wet weather flows. The SSES identified foundation drains as the primary cause of peak wet weather flows, contributing an estimated 50 percent of the maximum hour I/I flow.

System Description

The City is situated primarily on the uplands surrounding the flood plains of two rivers. The topography varies from level to slightly rolling, with numerous small streams creating natural drainage channels throughout the area. The soils are primarily silty clay loams with moderate to high permeabilities. The soils are characterized by sand lenses and glacial deposits. Annual precipitation is approximately 32 inches, with over 70 percent of the rainfall occurring from April through September. Groundwater levels are 0 to 5 feet below the surface during wet periods and 5 to 15 feet for much of the year.

The sewer system ranges from new to over 80 years old. Most of the oldest portion of system is concentrated in two of ten subsystems. About 40 percent of the collection system is over 30 years old, with pipes constructed with mortar or asphaltic joints. Pipes less than 20 to 25 years old (more than half of the system) are constructed with rubber compression joints. The sewer mains average about ten feet in depth, with lateral depths typically seven to nine feet.

Foundation drains were typically installed around new buildings beginning around 1945. Until 1962, it was typical practice for foundation drains to discharge through the basement wall directly adjacent to a floor drain. This provided direct entry into the house service lateral and thus to the sanitary sewer system. An ordinance adopted in 1962 required construction of a receiving sump inside the basement and discharge of the foundation drain water to a point outside of the building. Discharge to the sanitary sewer was prohibited. The usual practice was to discharge through a shallow plastic line to the street curb or yard. However, it is suspected that many foundation drain discharges were eventually re-diverted to the sanitary sewer.

The foundation drain tiles are typically placed about five to six feet below the ground surface. Of those that are connected to the sanitary sewer system, about 60 percent drain by gravity and 40 percent utilize a sump pump discharging to the sanitary lateral. In addition, there are many foundation drain sump pumps with normal discharge to the ground surface, but which are valved to allow discharge to the sanitary sewer in the winter during freezing conditions.

RII Documentation

The I/I Analysis and SSES field work included flow monitoring, smoke testing, dye flooding, manhole inspections, flow isolation, and TV inspection. Smoke testing and dye flooding were used to identify direct inflow sources and sources of indirect flow transfer from storm drains to sanitary sewers. Manhole inspections identified sources of direct inflow through manhole lids. In addition, a foundation drain study was conducted to provide documentation of the I/I flow contribution from foundation drains directly connected to the sanitary sewer system.

The foundation drain study included a survey to locate foundation drain connections, and measurements of foundation drain flows during rainfall and rainfall simulation. The survey included over 8,500 buildings and identified over 1,800 foundation drain connections to the sanitary sewer system. These included about 100 "wet basements" with no foundation drain, but where water flowing through cracks in the basement walls enters the sanitary sewer through the basement drain. In addition, another 1,600 foundation drain sump pumps with normal discharge to the ground were found. Many of these have valving capability to divert flow to the the sanitary sewer during freezing conditions. Presumably, if homeowners neglect to switch the discharge back to the yard at the end of winter, a portion of these foundation drains would also contribute flow to the sanitary sewer system during peak flow conditions in the spring.

Running time clocks were installed on 12 foundation drain sump pumps over a one and one-half year period. The locations were selected to provide a representative range of soil and groundwater conditions, and included locations where the sump pumps ran only during extreme wet weather periods, as well as locations where the foundation drain was active continuously except under extreme dry weather conditions. The data from the sump pump pumping study was used to project average flow rates for different design conditions. For the one-hour maximum flow condition, the average flow contribution per foundation drain was estimated to be 5.6 gpm.

Rainfall simulation was conducted for seven foundation drain locations. The testing was designed to simulate a 1 in/hr rainfall (estimated two-year recurrence frequency). For two of the sites in which the lots sloped away from the house, no response was detected and the testing was discontinued after 30 minutes. (These foundation drains were normally active during wet weather.) For five of the sites, an initial response was detected within about 10 to 20 minutes (less than 10 minutes at three of the sites). Of the four sites which could be measured, the average time to peak flow response was 45 minutes, with the peak flow averaging approximately 50 percent of the applied flow rate.

Based on the results of the SSES field work and foundation drain study, estimates were developed of the flow contribution from identified I/I sources. Direct inflow was estimated to account for approximately 40 percent of the maximum hour I/I flow, and foundation drains were estimated to contribute about 50 percent. The projected PWWF is estimated to be about six times the ADWF.

The highest flows in the system typically occur during the spring under high groundwater and saturated soil conditions resulting from successive rainfall events. The response of foundation drains was found to be faster and the peak flows higher under such conditions. City staff have observed that soil shrinkage may pull the soil away from foundation walls, which may be one factor in the rapid rate of infiltration into foundation drains. The factors that affect the response time and flow from foundation drains may include the amount and pattern of rainfall, soil type, soil moisture, groundwater, lot slope and construction, location of roof discharges, and landscaping.

RII Control Program

During the period since the SSES, the City has completed much of the rehabilitation work that was determined to be cost effective in the SSES analysis, primarily correction of direct inflow sources and some sewer rehabilitation. About two years ago, the City initiated a foundation drain disconnection program, targeted at eliminating 768 foundation drain connections over a ten-year period. The program includes provisions to reimburse a large portion of the homeowners' disconnection costs. To date, approximately 300 foundation drain connections have been eliminated, on an entirely voluntary basis. About half of these are in concentrated areas, with priority for disconnection being placed in areas with existing storm drain facilities. The City anticipates that the program will continue beyond the required 768 disconnections. No follow-up flow monitoring has been conducted, but City staff have observed a decrease in the number of complaints and basement backups.

Impact of Peak Flows on WWTP Operation

The existing WWTP, constructed in 1950, is a single-stage trickling filter plant with an design average flow capacity of 2.2 mgd. Although flows in excess of design capacity have been effectively handled at the plant, peak flows exceeded about 8 mgd must be bypassed. Current average flows are approximately 5 to 6 mgd, with peak hour wet weather flow rates estimated to be over 35 mgd. The current WWTP expansion will increase design capacity to 12 mgd average, 20 mgd peak day, and 34 mgd peak hour flow, utilizing a two-stage trickling filter/solids contact process and equalization basins to handle flows in excess of 20 mgd.

Generally, the existing plant can achieve 80 percent removal of BOD and suspended solids. During high flows, plant efficiency drops to 55 to 60 percent. Other problems which have been experienced during high flows include hydraulic washouts, carryover of solids, and digester upsets due to fluctuating solids loadings.

CITY OF COOS BAY, OREGON

The City of Coos Bay is located on the southwest coast of Oregon. The City is divided into two main sewer service areas, each served by a separate WWTP. The major I/I problems are concentrated in the collection system tributary to WWTP No. 1, which serves the eastern portion of the City and the adjacent Bunker Hill Sanitary District outside of the City. The Coos Bay wastewater system serves a population of about 15,000 and contains approximately 78 miles of sanitary sewers (not including tributary districts). The sewer system is primarily a separate system, although a small portion is believed to be partially combined.

Problems due to high peak wet weather flows include bypassing and overflows in the collection system, as well as raw sewage bypasses and discharge requirement violations at WWTP No. 1. In 1971, the City completed a comprehensive sewerage study, which identified I/I as a major problem in the collection system. From the early 1970's through 1982, the City conducted source detection and rehabilitation work to reduce I/I. The program included disconnection of known direct inflow connections, including downspouts and cross connections with the storm drain system, as well as sewer main rehabilitation. However, despite the rehabilitation program, the system still experiences high peak wet weather flows. Since smoke testing has confirmed that almost all direct inflow connections have been eliminated from the system, it is believed that the majority of the peak rainfall induced flows are due to infiltration into sewer main and service lateral defects.

System Description

The City is located on a peninsula surrounded by Coos Bay, the largest estuary in Oregon. The two sections of the City are situated on the eastern and western sides of the peninsula, corresponding to the WWTP No. 1 and WWTP No. 2 service areas, respectively. The topography is characterized by rolling foothills, with elevations varying from sea level to 500 feet. The flatter areas are located near the edges of the estuary. Soils are marine and alluvial deposits, primarily sandy loams with greater amounts of silt and clay in the eastern (WWTP No. 1) area, including bay mud in the downtown area near the estuary. A large portion of the older area of the City is located in a tidal basin and constructed on dredge spoils (fill). Average annual rainfall is approximately 62 inches, with 75 percent of the rain falling during the period November through March. Groundwater elevations near the estuary are very high and influenced by tidal fluctuations. In other areas, the groundwater level is typically below the sewers for most of the year, but increases during the winter rainy season.

The sewer system was originally constructed in the central portion of the City (formerly called Marshfield) around 1925. Prior to 1954, the system consisted mainly of combined storm/sanitary sewers. In 1954, when the original WWTP No. 1 was constructed, most of the sewers in the original part of the City were separated, although many storm drain/sanitary sewer interconnections were still found to exist during the 1971 sewerage study. These interconnections were removed, including construction of new storm sewers where required, during the 1971 through 1982 rehabilitation program. Substantial development occurred in the system during the 1950's and 1960's.

The sewer system is composed mostly of concrete pipe, but also includes VCP, AC, CMP, and some plastic. The sewers in the older portions of the City are primarily VCP and older, more porous concrete pipe. Joints are mortar or rubber ring, depending upon the date of construction. Over half of the system is composed of "older" pipe materials. Sewer depths average eight to ten feet; laterals are typically about three feet deep near the building, dropping down to the depth of the sewer in the street.

RII Documentation

Dry weather flow monitoring at 34 locations and wet weather flow mapping (early morning flow measurements taken at key manholes) were conducted as part of a Facilities Plan Supplement completed in 1987. The difference between wet weather and dry weather flows was defined as the rainfall dependent flow, and ranged from zero to 56 gpd/ft for the 34 subbasins. The projected peak rainfall dependent flow for a five year design storm was calculated, resulting in a projected PWWF of about 14 mgd and a PWWF to ADWF ratio of about 8 to 1.

Extensive smoke testing was conducted during the period 1972 to 1975 as part of the sewer system rehabilitation program. Although the primary objective of the smoke testing was to locate direct inflow sources, many leaking service laterals also exhibited smoke. TV inspection identified problems in sewer mains due to leaking joints and root intrusion. In areas near the estuary, ground settlement has caused considerable pipe movement, resulting in cracks, breaks, and offset joints in the sewers and service laterals.

Based on the previous elimination of all identified direct inflow connections and the known poor condition of the sewer system, it can be concluded that the peak wet weather flows in the Coos Bay system are primarily due to RII.

RII Control Program

In previous years, the City has completed rehabilitation (primarily grouting and some replacement) of sewer mains with major problems identified through smoke testing and TV inspection. No work on service laterals has been conducted. Although the basic approach to addressing the wet weather flow problem in the system consists primarily of expansion of the WWTP, the City has instituted a program of routine TV inspection of the sewers to identify particular areas in need of repair or replacement.

Impact of Peak Flows on WWTP Operation

The existing Coos Bay WWTP No. 1 is a conventional activated sludge treatment facility with a design average flow of 2.66 mgd and a maximum hydraulic capacity of 5.85 mgd. In addition to raw sewage bypasses, the biological process is frequently upset by hydraulic overloading, resulting in solids washouts. A split stream treatment scheme is practiced, which provides primary treatment with disinfection to all plant influent flows and secondary treatment up to process design limits. This practice has been relatively successful at meeting effluent discharge limits except during extreme high flow conditions. Salt water shock loads that upset digester operation have also occurred at the plant, believed to be due to salt water inflow into storm sewers through malfunctioning tide gates and subsequent infiltration into the sanitary sewer system. The treatment plant is currently being expanded to handle the projected five-year storm design peak hour flow of 14 mgd utilizing a similar split-stream process scheme during peak flow periods.

CITY OF TULSA, OKLAHOMA

The City of Tulsa is located in northeast Oklahoma along the Arkansas River. The wastewater service area is divided into two main drainage basins, referred to as the Northside and Southside systems. The total service area population is approximately 400,000, and the collection system includes over 1,400 miles of sewer mains.

The City has conducted SSES work in the sewer system since 1982 as part of overall facilities planning efforts and in order to reduce surcharging and overflows in the collection system during rainfall. In subbasins determined to have excessive I/I, SSES work has been followed by rehabilitation. In the Southside basin, SSES work was completed in 36 of 41 subbasins, and rehabilitation has been completed or designed for 17 subbasins. In the Northside basin, SSES field work was conducted in 12 of 22 subbasins and rehabilitation in 8 subbasins. Source flow estimates based on the Northside SSES indicate that over 70 percent of the peak rain-induced I/I flow is contributed by infiltration sources in collection sewers, manholes, and service laterals.

System Description

The topography of the service area ranges from flat to gently sloping. The area is generally characterized by shallow bedrock ranging from 20 to 60 inches below the ground surface. The bedrock consists mainly of limestones, shales, and sandstones, overlain by moderately to well-drained loamy soils formed from materials weathered from the bedrock or from alluvial deposits. In the eastern portion of the service area, the soils are typically tight, expansive clays. Average annual rainfall is about 39 inches; the highest rainfall occurs during the months of April through June and September and October. Groundwater levels are typically low. Only scattered areas experience high groundwater, typically near the rivers, and the maximum seasonal groundwater levels are generally not higher than about six feet below the surface.

The oldest portions of the sewer system date to the early 1900's. About one-half of the existing system was constructed before 1960, and the system has continued to expand through the 1980's. The sewers in the system are predominantly VCP, with some concrete and plastic pipe. Cast iron or PVC laterals are most common, with VCP, AC, concrete, and Orangeburg pipe in the older areas. The older pipes in the system utilized tar, jute, or cement mortar joints and were bedded and backfilled using native soil materials. Newer pipes have been installed with sand bedding, and since 1962 have utilized molded plastic or rubber gasket joints. Manholes are predominantly brick and mortar construction, with precast concrete manholes being installed more recently in some of the newer areas of the system. In general, the manholes do not have vent holes. The sewers mains average about ten feet deep and are generally located above the groundwater. Service laterals are typically two and one-half to six feet deep. About 90 percent of the sewers are located in backyard easements or alleyways.

RII Documentation

Flow monitoring was conducted at 24 sites in the Northside basin and 46 sites in the Southside basin to determine dry weather flows and rainfall induced I/I. In both systems, dry weather (non-rainfall) infiltration was not found to be excessive, which is consistent with the low groundwater levels in the service area. Measured PWWF to ADWF ratios typically ranged from 2 to 5, with a few subbasins experiencing higher peaks. For the overall Northside and Southside systems under projected design storm conditions, the PWWF to ADWF ratios are about 3.5 to 1. The rainfall induced I/I represents about 70 percent of the peak wet weather flow.

The field investigations conducted as part of the SSES's included extensive smoke testing, as well as dye flooding, manhole inspection, and TV inspection. Based on the data collected in the eleven SSES subbasins in the Northside basin, the predominant types of I/I sources identified were leaks from service laterals, sewer mains, cleanouts, and under manhole frames. Over half of the defects were detected as smoke returns observed along the ground over service laterals and sewer mains. There were very few direct inflow connections such as roof leaders, area drains, or storm drain/ sanitary sewer cross connections.

Visual and TV inspections were conducted to assess the overall condition of the system. The TV inspection data was used primarily to determine appropriate rehabilitation methods, and not to specifically identify or quantify I/I sources. Common deficiencies observed during these inspections included offset joints, cracks, and root intrusion. Lateral taps were also found to be a significant problem.

Rainfall simulation (water and dye water flooding) were conducted for portions of the SSES areas to evaluate leaks from sources identified through smoke testing and manhole inspections. These procedures consisted of flooding the surface in the area around the suspected I/I source (leaking sewer, service lateral, or manhole) and measuring the rate of leakage. For manholes, the location of the leak (at the cover, under the frame, or through the wall) was also noted.

Based on the results of the rainfall simulation, leakage rates, either measured or estimated, were assigned to each of the I/I sources identified through the SSES field work. For the Northside SSES subbasins, identified RII sources (leaks from sewer mains and service laterals, through manhole walls and cleanout risers, and under manhole frames) account for about 70 percent of the total quantified rain-induced flow, and inflow sources (direct surface drainage connections and leaks through manhole or cleanout covers) account for about 30 percent. Approximately 45 percent of the identified RII appears to be due to service lateral sources, 35 percent from sewer mains, and 20 percent from manholes.

The identified RII sources, however, appear to account for less than half of the total wet weather I/I determined through flow monitoring. The remaining rainfall induced flow may be due to RII sources in sewer mains and laterals that could not be detected through smoke testing. City staff identified the French drain effect of granular backfilled trenches in areas where the soils are tight or the bedrock shallow as a potential factor in causing RII in the system. In areas with shallow limestone bedrock, blasting has also caused damage to the pipes.

RII Control Program

Rehabilitation was performed as part of the SSES work. The rehabilitation work consisted primarily of slip-lining, inversion lining, pipe replacement, manhole sealing, spot repairs, and disconnection of direct inflow connections. In general, only those defects detected through SSES field work and determined to be cost effective for correction were addressed. Therefore, the rehabilitated areas typically constituted less than 10 percent of the total sewers in each subbasin and were generally not contiguous. In those areas where slip-lining was done, the lateral connections were replaced and the portion of the lateral within the easement was also slip-lined or replaced.

The rehabilitation projects included a public relations program to encourage voluntary repairs of private laterals by property owners. Property owners with laterals or cleanouts indicated to be leaking based on the SSES tests were notified and requested to make repairs. An overall 70 to 80 percent compliance was achieved in many areas.

Flow monitoring was performed before and after rehabilitation in order to assess the overall effectiveness in reducing I/I. For eight Northside subbasins in which rehabilitation was performed, the initial reductions in peak wet weather flows ranged from approximately 30 to 90 percent, with an average of about 50 percent.

Impact of Peak Flows on WWTP Operation

The City is served primarily by two major WWTP's, Northside and Southside. Both are conventional activated sludge plants, each treating an average dry weather flow of about 30 mgd. At the Southside WWTP, the peak flow reaching the plant is limited to 50 to 60 mgd by pump station capacity. Flows in excess of this amount are bypassed to the Arkansas River. The Northside WWTP has limited equalization storage capacity; however, it is insufficient for handling peak wet weather flows. All flows which enter the plants pass through all treatment process units. During very high flows or prolonged high flow periods, washouts can occur in the secondary treatment process.

APPENDIX D

SEWER SYSTEM REHABILITATION METHODS

This appendix contains discussions on sewer system rehabilitation methods applicable for control of RII. The appendix is divided into three sections: Pipeline Rehabilitation, Manhole Rehabilitation, and Foundation Drain Disconnection

PIPELINE REHABILITATION

Rehabilitation methods for sewer pipelines are divided into two categories: (1) replacement by conventional and trenchless techniques, and (2) rehabilitation by grouting and by lining techniques, including slip-lining and cured-in-place lining. The methods described in this section are not all-inclusive; other techniques are currently being developed.

The focus in pipeline rehabilitation today is on in-place techniques such as trenchless pipe replacement, slip-lining, and cured-in-place lining. These methods minimize the impact at the surface, for example, minimizing traffic disruption and conflicts with other utilities. One of the main shortcomings of all the in-place techniques is making a leak-free joint at the main and lateral connection without excavating. Because these connections are often significant sources of leakage, the effectiveness of the seal at this joint may be essential to RII reduction.

Many rehabilitation techniques orginally developed for sewer mains have been modified for lateral rehabilitation. However, the cost effectiveness of these methods generally becomes less as the length of the individual rehabilitated pipe decreases. Since laterals are typically short (less than 75 feet) and may have many bends or offsets, rehabilitatation of laterals by in-place techniques is generally less cost effective than for mains. Access to laterals for both testing and rehabilitation also remains a technical and institutional problem.

Replacement

Replacement is an effective option for RII correction, as well as for repair of structural deficiencies. Replacement of an entire manhole-to-manhole reach initially provides a new, essentially leak-free pipe. Conventional replacement methods involve excavation and removal of the existing pipe or excavation of a parallel trench for the new pipe with abandonment of the existing

pipe in place. Excavation and replacement of isolated, joint-to- joint pipe sections (point repairs) may also be used as a means of RII source correction, or in conjunction with other sewer
rehabilitation techniques. Whereas in-place repair techniques
leave the original pipe grade, offsets, and sags unchanged, excavation and replacement generally correct these types of problems, as well as more severe problems such as sewer collapses. Excavation for point repairs are often necessary with other rehabilitation techniques. For example, severe joint offsets must be excavated and repaired prior to slip-lining. Lateral to main sewer connections are also frequently excavated for repair.

Some of pipe replacement techniques that do not require excavation are described in the following paragraphs.

Tunneling. There are a wide variety of construction techniques that can be classified as tunneling. These techniques include microtunneling and auger boring as well as conventional tunneling. Tunneling is a means of replacing an existing pipe without extensive excavation.

Microtunneling refers to tunnels which are too small to allow man entry. Microtunneling techniques are also varied, and many are steerable. In general these techniques allow installation of pipe up to a maximum of 300 to 400 feet. The minimum pipe size for today's equipment is about 8 inches. Techniques are available for installation below groundwater. The technique is generally applicable in silts, clays, sands, and gravels, but deals poorly with stones or cobbles larger than two inches. The accuracy of the installation depends in part on the length. Concrete, clay, and fiberglass reinforced plastic pipe have been installed using this technique. Access requires excavation of a launch pit 12 to 20 feet long. Microtunneling can also be used to replace an existing pipe along its alignment. The technique has been used extensively in Japan, Germany, France, and Singapore.

Auger boring is used to describe a nonsteerable technique. Accuracy and length of installation are less than for microtunneling. Pipes as small as six-inch diameter with lengths up to 400 feet can be installed. Access requires pits at each end, 6 to 15 feet in length. The pipe installed is generally steel and called a casing. The process or carrier pipe is slipped inside the larger steel casing. This technique, often referred to as boring and jacking, has been used since the 1940's in the United States. Pipe jacking is a variation of the bore and jack technique where the casing is eliminated and the pipe itself is jacked.

Moles. Moling involves use of a percussive hammer to create a duct through the soil. There are two major variations, impact moling and pipe bursting (also referred to as pipeline insertion machines). These techniques allow for trenchless installation of new pipe. The mole is generally non-steerable. Pipe bursting depends on an existing pipe for alignment.

Impact moling typically utilizes a percussive hammer driven by air, although hydraulic versions have been developed. The mole creates a duct through clay, silts, sands, and gravels. Isolated boulders or cobbles can be broken but often throw the mole off its alignment. Very soft soils do not provide enough support for the weight of the mole; the mole may drop making reasonable grade control impossible. Normal installations are 100 feet long but installations up to 200 feet long have been made. Steerable moles exist; however, most moles depend on the initial orientation for their alignment. Pipes between one- and six-inch have been installed with this technique. The size of the launch pit is generally determined by the length of the pipe sections installed; the mole itself requires a launch pit of about six feet in length. This technique is widely used in the United Kingdom to install individual services.

Pipe bursting uses an expander in conjunction with a conventional impact mole. The expander, larger than the existing pipe diameter, breaks the pipe and allows a new pipe to be pulled or pushed into the space behind the expander. The pipe installed may actually be larger than the existing pipe. Pipes up to 18-inch have been installed by this technique. The maximum length of installation is about 450 feet. Pipe bursting is effective in existing cast iron, unreinforced concrete, clay, and asbestos cement pipes. New polyethylene, polypropylene, and clay pipes have been installed by this technique. Butt welded polyethylene is particularly attractive for rehabilitation for RII control, since it is jointless. A launch pit about 10 feet in length is required, although a longer launch pit may be required depending upon the type of pipe installed.

Moles are frequently utilized for laterals, both for new construction and rehabilitation. Moles can be used both inside existing pipes for replacement (rehabilitation) and for installation of a new pipe. Impact moling provides a number of advantages. Parallel replacement of a lateral can allow the existing lateral to remain in service until the new service is installed. Also, new construction using impact moling installation does not require granular backfill, thereby minimizing the potential for infiltration into the sewer trench.

Rehabilitation

Rehabilitation (as opposed to complete replacement) of an existing pipe can be accomplished "in-place" by several methods, including grouting slip-lining, and cured-in-place lining. These methods are discussed below.

Grouting. The least disruptive technique for rehabilitation, grouting, focuses on the sealing of joints, small holes, and radial cracks in otherwise sound pipe. This technique involves no excavation where manhole entry is available. Grouting is performed with a miniature television camera which locates the pipe joint and defects. Air testing may be used to determine which joints are leaking and therefore require grouting. After positioning, a temporary, double-bladder seal isolates the joint and grout is pumped through the joint. After grouting, the joint is pressure tested to ensure the adequacy of the seal.

A variety of chemical grouts are available. The chemical grouts include acrylamide gel, acrylate gel, urethane gel, and polyurethane foam. The gels are all capable of penetrating the pipe joint and filling voids outside the pipe wall. The foam simply forms a gasket in the pipe joint.

The longevity of grout sealing may vary. Some of the grout products are susceptible to shrinkage under alternate wet and dry cycles (such as when the groundwater level varies above and below the pipe), reducing their sealing effectiveness. Foam grouts are designed to be unaffected by water conditions, but may be difficult to apply. In all grouting, quality control during application may have a significant impact on grouting effectiveness. Therefore, periodic testing after the initial grouting (e.g., every three to five years) may be required, not only to re-test the seal on grouted joints but also to ensure that new leaks in previously ungrouted joints and defects are also addressed.

Lateral Grouting. Specific grouting techniques have been devised for lateral rehabilitation. One technique involves pumping the lateral full of grout under pressure and then cleaning the excess grout from the pipe interior. This method of joint sealing has shown limited success.

A second method known as the "sewer sausage" for grouting laterals has been devised. In this method, an inflatable plastic sock is inserted into the lateral through the main sewer using a special device that is operated by remote control. The device is located inside the sewer at the lateral, and is controlled by the operator by viewing through a TV camera. The inflatable plastic sock generally covers the first two to three joints. Grout is injected under pressure into the annulus created by the sock. After the grout is set, the plastic sock is pulled out and moved to the next lateral in the sewer. Although the sewer sausage method has been used for entire laterals, it is most effectively used for sealing the first joint between the lateral and the main sewer.

In addition to the above techniques, direct joint grouting techniques, as used in larger diameter sewers can also be utilized for laterals.

Slip-lining. In slip-lining, a liner pipe, slightly smaller in outside diameter than the inside diameter of the existing pipe, is inserted into the existing sewer. Prior to installation, the existing pipe must be televised to identify potential obstructions such as severe offset joints and protruding laterals, and failed pipe sections. These must be corrected by point repair. Televising also serves to identify the locations of services which must be connected to the completed slip-lining. Proofing the pipe by pulling a short piece of liner through the pipe is recommended.

The slip-line insertion process involves excavating a small length of existing pipe to provide an insertion pit. The depth and size of the excavation depend on the depth, diameter, and the flexibility of the pipe liner. The liner, most often high-density polyethylene, is flexible and can be butt fused into long joint-free sections on the ground surface. The slip-lining pipe is pulled by a steel cable and is oftentimes assisted by pushing the lining into the existing pipe. A tapered, pulling head provides gradual size transition and prevents debris from entering at the leading end. The gradual size transition makes it possible to pass minor obstructions.

The ends of the liner at the manholes typically are grouted to seal the annular space between the liner and the outer pipe. Full grouting of the annular space may also be done. This decision is generally based on cost, the condition of the existing sewer, depth of cover, the potential for point loads on the pipe, and the amount of groundwater present. The slip-lining is resistant to attack from acid, such as sulfuric acid commonly formed from hydrogen sulfide in sanitary sewers. This characteristic makes slip-lining suitable for repair of sewers with high potential for corrosion.

When a sewer main is slip-lined, each lateral connection must be excavated and reconnected to the slip-lined pipe. If the lateral is also slip-lined, the lateral and main sewer liners can be fused together to make a leak-free joint.

Two variations of the conventional slip-lining method are now available, both of which can be installed from existing manholes and therefore eliminate the need for excavation for insertion pits. The first method utilizes short, threaded high density polyethylene (HDPE) pieces. The physical properties of this material are higher than polyvinyl chloride (PVC), while its resistance to chemicals and effect of temperature on physical properties are similar to that of PVC lining. The assembled liner pipe can either be pushed or pulled with the existing pipe by using simple winching equipment. Low density grout is used to fill the space between the pipe and the lining. The cost of sliplining with short pieces may be less than other systems because non-skilled labor can be used for installation.

The second non-conventional slip-lining method uses a specially designed PVC strip that is spun directly into the existing sewer to be rehabilitated. The PVC strip is helically wound by a machine placed in an existing manhole. The space between the pipe and the lining is filled with low density grout to stop groundwater from leaking into the annular space. The PVC lining used in this process has excellent properties as a protective lining against corrosion and can be designed for any strength requirements.

In another variation of this method, the lining is expanded after insertion into the existing sewer. This is accomplished by pulling an inflated plug through the liner in the sewer while spiral joints slip before the cement is set. With this method, no grouting is required since the lining touches the pipe. However, a bonding resin is recommended to be used between the lining and the pipe.

As with conventional slip-lining, there is no dependable remote control method for cutting the internal connections. The connections must be excavated and exposed, the liner pipe cut, and a fabricated connector fitted and adhered to the lining with solvent cement. The entire fitting is then covered with cement mortar.

Expandable plastic liners (polyethylene and PVC) are recent developments in lining of pipe. These liners come in flattened rolls. They are heated slightly as inserted to increase flexibility. After installation, further heating results in reversion to the original circular cross section. Handling of manhole and lateral connections is similar to that for other slip-lining methods. These products have been through limited actual usage.

Cured-In-Place Lining

Cured-in-place lining techniques utilize a thermal-setting, resin-coated, flexible fabric, which is prepared to match the diameter and the length of the pipe section to be lined. The material is saturated with resin and kept chilled prior to installation. Once in place, the liner is cured and hardened. The liner conforms to any shape and discontinuities and provides a smooth, joint-free lining. The liner thickness is a design choice; thicker linings can be designed to support weakened pipe or support greater hydrostatic loads.

Until recently, only one cured-in-place lining method was available in the U.S. However, several other methods have been developed and are in use in Europe and Japan. At least one of these has recently entered the U.S. market. All cured-in-place lining methods use the same basic materials (thermo-setting resins), but differ in the techniques used to insert the liner into the pipe and the method for curing the resin. In most methods, the lining is inserted into the existing pipe by inversion, although in at least one method the lining is slipped inside the pipe. Inversion is generally accomplished with water or compressed air. Different methods may be used for curing the liner; hot water or steam are the most common, but ultraviolet light is used in one process.

Cured-in-place lining does not require any excavation unless there are major pipe failures or severe lateral protrusions into the existing line. The pipe material is resistant to acid and can be used to repair corroded concrete sewers. Although a remote cutting device can sometimes be used to reconnect laterals to the lined pipe, the exact location of the lateral connections may sometimes be difficult to find. If the original lateral connections are subject to leakage, remote cutting will not provide any means of sealing these joints. In such cases, the lateral connections would have to be excavated and repaired.

MANHOLE REHABILITATION

The magnitude of RII through manhole defects appears to vary widely from system to system. It is well known that inflow through manhole lids can contribute to peak wet weather flows, particularly when manholes are located in areas subject to ponding or flooding. Less well documented is the RII through manhole defects where pavement defects allow rain to move quickly into base rock materials adjacent to manholes. Milwaukee Metropolition Sewerage District (MMSD) studies suggested significant RII flows can result from manhole defects, specifically, frame and chimney connections. RII may also enter manholes through the walls and base, particularly in brick manholes with deteriorated mortar.

Rehabilitation methods for manholes include both interior and exterior techniques. Interior repair techniques are less expensive and less time consuming than external repairs but are frequently less effective. MMSD has conducted a pilot testing program on the various manhole rehabilitation techniques described below, as well as others deemed ineffectual; they continue to evaluate the effectiveness of the arious techniques.

Interior Repairs

Interior repairs are typically less effective for infiltration control but remain attractive in many cases due to the low cost and ease of construction. These techniques make possible the sealing of all manhole joints including the lower ones, which are often subject to the largest hydrostatic forces. Interior repair techniques utilize elastomeric sealant, chemical grout, or an internal boot.

Sealants. Internally applied sealants are intended to seal leaky joints in the manhole wall including the manhole frame and chimney joint. The surface must be free of loose material, gaps filled, and the surfaces must be cleaned to assure a bond for the sealant. Various products are available to be either troweled or brushed. A potential disadvantage of elastomers is that hydrostatic pressure can destroy the bond, requiring replacement in the future.

Grouts. Grouts may be utilized to plug voids behind manhole walls much as they are used in pipeline rehabilitation. The chemical gels have not functioned well in applications where alternate wetting and drying occur. A grout that is not subject to this complication could provide a positive seal since hydrostatic forces would not destroy the sealing capabilities.

Internal Boot. An internal boot utilizes a continuous band of elastomeric material forced against the manhole walls with adjustable expansive metal bands to seal manhole joints. The boot provides for vertical displacement at the joint but has limited offset capabilities. The concrete contact surface must be smooth and without ridges which might preclude a seal.

Exterior Repairs

Exterior repairs are often more effective than internal repair methods, but require excavation. Therefore, external manhole repair methods are more costly more than internal repairs. It is difficult to gain access to all the manhole joints, consequently repairs focus on the joints close to the surface. These techniques utilize elastomeric sealant, elastomeric sheet, rubber sleeves, and two-piece frames.

Elastomeric Sealants. These elastomeric compounds are poured around or troweled on the manhole joint. The poured versions are available in cold pour and hot pour mixtures both requiring a form to contain the pour. The cost of both techniques are similiar since the excavation, backfill, and pavement repair costs are significant percentages of the total cost. MMSD found the trowellable version to be most attractive.

Elastomeric Sheeting. Elastomeric sheeting can be banded or applied with adhesive to the outside of the manhole structure. Joints in the sheeting may be thermally welded when thermoplastic materials are utilized.

Rubber Sleeves. Rubber sleeves similiar to the internal boot are manufactured. These can be slipped over the manhole chimney. The sleeves are held in place by upper and lower stainless steel band clamps. Some versions are designed to accommodate vertical movement in between the manhole frame and chimney.

Two-piece Frames. Two-piece frames provide another means of achieving vertical flexibility. The lower section of the frame is securely anchored to the top of the manhole chimney and will not be displaced by surface movement; therefore, the frame/chimney joint remains intact. An elastic, water-tight gasket provides flexibility for vertical movement and a seal against infiltration between the two pieces of the frame. Since the upper portion of the manhole frame must be supported by the pavement, a two-piece frame is probably only suitable for application in rigid concrete pavement.

FOUNDATION DRAIN DISCONNECTION

Foundation drains may be a significant RII source in some areas, as shown by the case studies presented for MMSD and Ames, Iowa. Many cities have ordinances that prohibit direct connection of foundation drains to sanitary sewers. However, many older installations still exist. It is also not uncommon for foundation drains to be diverted to sanitary sewers because of accumulation of ice or water outside the building or in the street when the discharge is not connected to a storm sewer.

Foundation drains may be connected to sanitary sewer laterals in one of several ways:

- o Direct gravity connection to the sanitary lateral.
- Drainage into a sump with a pumped discharge to the sanitary
- Drainage into a sump with a gravity discharge to the sanitary
- o Discharge onto the basement floor and drainage to a basement drain connected to the sanitary lateral.

Methods for foundation drain disconnection are relatively straightforward. Depending upon which of the above existing configurations apply, the foundation drain discharge is directed to a sump (if one does not already exist), and a sump pump and discharge line are installed (or the existing sump pump discharge is redirected) to the ground surface outside of the building or to a storm drain. Discharge of the foundation drainage to the storm drain system would also require connection to an existing storm lateral or construction of a separate storm lateral to connect into the storm sewer. The connection to the sanitary sewer lateral must also be plugged. It is important to make sure that sanitary sewage cannot enter the storm sewer, and that basement floor drains are connected to the sanitary sewer.

APPENDIX E

DESIGN AND CONSTRUCTION STANDARDS

This appendix contains a discussion on design and construction for minimizing RII into new sewer construction.

DESIGN STANDARDS

Modifications to sewer design standards provide a means to minimize future RII in new construction. Such modifications include: restricting the flow of water in granular backfill, reducing interconnections between backfills of various utilities, reducing the number of pipe joints, providing flexibility to reduce settlement stress and breakage, sealing pipe/manhole connections, and control over lateral installations.

Restricting Flow of Water in Granular Backfill

Granular backfill in pipe trenches can dewater surrounding soils with resultant increased settlement potential for the pipe and the ground surface. This results in stress and the potential for creation of RII entry points in the buried pipeline. Furthermore, the granular backfill provides a permanent hydraulic conduit (French drain) along the exterior of the pipeline. This hydraulic conduit can provide the means for large quantities of water to travel to damaged joints and pipe defects.

This phenomenon can be alleviated by the addition of impermeable trench cut-off walls, or trench plugs. The trench plugs consist of concrete, grout with cement, or bentonite clay to create an impermeable dam. The number of trench plugs needed depends upon the slope of the pipe (and the backfill); trench plugs at more frequent intervals should be specified for higher slopes. An interval of 50 feet is common for such trench plugs.

If possible, a connection should be provided between the backfill and a point where the collected water can be discharged. Such connections could be to a storm drain or a creek. It is ideal to provide these connections at trench plugs at an elevation below the spring line of the sewer.

In some areas, other methods to reduce the permeability of the granular backfill have been tried. These include the introduction of impermeable grouts into the granular backfill after placement, the inclusion of additives such as cement or bentonite clay to granular backfill, or the specification of a well-graded backfill material. These measures serve to retard the rate at which water can infiltrate into sewer trenches.

Reduction of Utility Backfill Interconnections

A related issue is the common practice of placing granular backfill in areas where utilities cross. This is done because adequate compaction is difficult to achieve when utility trenches are in close proximity, either vertically and horizontally. Granular backfill (e.g., pea gravel) can be compacted to higher levels with less compactive effort. Unfortunately, this backfill material also provides a pathway for water collected in the shallower utility trenches to move into the backfill surrounding the sanitary sewer, almost always the deepest utility. Trench plugs can be installed at these locations to prevent this connection.

Control of Migration of Fines/Piping

Sewers constructed at steep slopes and in areas where groundwater is constantly fluctuating present the problem of migration of soil fines. This migration can take place along the pipe and at cross section to the pipe.

Two separate measures should be considered in such situations. Installation of trench plugs may prevent the fines from being carried away downstream. Installation of a semi-permeable membrane below and around the backfill may prevent soil migration in and out of the trench. These measures will benefit in reducing pipe subsidence and the subsequent formation of cracks and openings in the pipes. Also, the migration of fine particles away from the pipe trench will be discouraged, resulting in more resistance to RII movement within the trench and into the sewer pipes.

Reduction in the Number of Pipe Joints

The use of pipe with fewer pipe joints is advantageous since the joints are potential RII sources. Old vitrified clay pipe used in the past for sewer mains and service laterals had joints as close as two feet apart. Early joints were mortared and were subject to cracking and deterioration. New pipe materials, such as polyethylene, PVC, or ABS pipe can provide almost jointless construction. Fewer joints simplify the determination of the source of problems indicated by failed performance tests following construction. Fewer joints are also likely to reduce the number of problems with roots growing into pipe joints.

Flexibility to Reduce Settlement Stresses

Stress points occur at the connections of the main and lateral, the lateral to the house, manholes, cleanouts, and other structures. Stress points may also occur in trenches where underlying soil conditions change. The ability to accommodate differential settlement is important since unless the pipe transfers a part of the overburden soil load to the supporting soil, the pipe must carry the entire load. Flexible connections may be provided by two joints in close proximity as well as by flexible materials such as rubber couplings. Flexible connections are important between laterals and sewer mains, since these connections are often documented as major sources of leakage.

Manhole Connections/Joint Sealing

Sealing pipe connections at manholes is equally as important as providing flexibility. Manholes generally have greater hydrostatic pressures outside the manhole than within. Most manholes have no seep ring or water stop around the pipe as it enters the manhole. Many of the pipes used today, such as PVC and polyethylene, do not bond well to concrete manholes. Some additional means of sealing the pipe connection to the manhole is required to prevent infiltration into this joint. Rubber seals have been developed for small pipes. Tape seals, which are composed of bentonite and butyl rubber mixtures with adhesive backing, wrap around the pipe to form an expansive seal.

Laterals

Laterals are extremely important because they may represent about one-half or more of the total length of collection system piping. RII in laterals has been shown to be very high in many areas. This is due in part to lack of design and construction standards for laterals, limited degree of construction inspection normally provided, and because laterals typically receive little or no routine maintenance. Exterior cleanouts allow ready access for testing; one two-way connection at the street (property line) and one at the building is ideal. To minimize RII, each lateral connection at the main should be closely inspected, and the connection should utilize a manufactured sanitary tee, wye, or a saddle. Pipe penetrations (hammer taps) should be replaced. Flexibility can be provided as described earlier. Cut-off walls or trench plugs in the lateral trench can be an element of construction, particularly here grade change or lateral length is great and granular backfill is utilized.

CONSTRUCTION STANDARDS

Construction standards imply conformance to the design intent. This conformance is accomplished by inspection and testing.

Regular Inspection

Stringent construction standards for sewer lines cannot be realized without adequate inspection. Major sewer construction should be continually monitored. Although this would be ideal with laterals, it is impractical to provide more than a periodic inspection during construction. Many agencies require post-construction television inspection of new lines and laterals. This is quite valuable, but is not a substitute for inspection during construction. For example, post inspection viewing does not indicate if adequate compaction has been performed, if trench plugs have been installed, or if flexible couplings have been provided. Ordinances requiring strict compliance with standard construction details may help.

Performance Testing

Stringent standards for leakage testing (air pressure or water) should be set and achieved. Since the results at the time of testing are probably the best the pipe will ever achieve, stringent test standards are necessary to assure acceptable infiltration over the life of the pipe.

Leakage testing rarely imposes limitations on the length of pipe to be tested at one time. Anything shorter than manhole to manhole testing is impractical. However, since permittable leakage is a function of length, longer reaches allow greater losses. Current standards permit some joints to leak and this is practical to accommodate construction capabilities. However, one joint may be responsible for 90 percent of the leakage in a test section. If the test includes two joints, and one is the are badly leaking joint, the test would fail. If the test includes 40 joints, and badly leaking joint is included, the test may pass. Although testing of individual joints would eliminate this problem, it could be costly and time consuming.

With leakage tests, no pipe lengths greater than single manhole-to-manhole reaches should be tested at one time. Testing of individual joints is recommended in large diameter piping, 18-inch and larger, using joint testing equipment.

Criteria for exfiltration and air testing for gravity sewers and laterals are presented below. Test criteria should be modified according to the manufacturer's recommendations.

Exfiltration Test Criteria. Maximum allowable leakage of 25 gallons in 24 hours per inch-diameter-mile of sewer is recommended by some manufacturers. The Standard Specifications of Public Works Construction prepared by the County Sanitation Districts of Los Angeles County recommend the following formula for maximum allowable exfiltration.

$$E = 0.0001\, LD\, (H)^{1/2}$$

Where

E = Allowable leakage (gpm)
L = Length of test section (feet)
D = Internal diameter of pipe (inches)

H = Difference in elevation between water surface and invert at lower end of pipe (feet)

Air Test Criteria. When testing a new pipe, the common procedure is to maintain air pressure at 3.5 psig while the temperature stabilizes. The system passes the test if loss of pressure is 0.5 psig or less in 30 minutes. Failure to hold air pressure is usually indicated within 15 to 30 seconds.

APPENDIX F

COST EVALUATION

This appendix describes the methodology and results of the RII cost evaluation summarized in Chapter 3.

ANALYSIS METHODOLOGY

The approach utilized for the RII cost evaluation is primarily intended to address sewer systems, such as the EBMUD system, where the primary sources of RII are defects in sewer mains and laterals. It is not intended for systems in which the primary RII sources are manhole frame/chimney leakage, foundation drains, or other specific types of sources not generally classified as pipe defects.

The approach also assumes that the primary component of the peak I/I flow is RII. In particular, it is assumed that base groundwater infiltration (GWI) is not "excessive" (as defined under current EPA regulations) and that direct storm water inflow (SWI) is insignificant compared to RII peak flows.

The cost analysis procedure is intended to be applied to a sewer subsystem which is relatively homogeneous with respect to age, soils, geology, groundwater conditions, sewer depths, and the general physical condition of the system. A typical application would be for a monitored area of between 10,000 and 50,000 linear feet of sewer mains. It is assumed that the RII flows for the subsystem have been previously determined by flow monitoring or that a reasonable estimate of the RII can be made. As discussed in Chapter 2, the magnitude and pattern of RII flows are a function of many different interacting factors. Therefore, the RII response cannot necessarily be predicted for any particular area based solely on the physical characteristics of the area or the sewer system.

The assumptions used in the cost analysis should not be perceived as limiting its applicability to more "realistic" situations, for example, where GWI is also a significant flow component. The basic concepts and approach can be applied to more complex situations with appropriate modifications.

The RII cost analysis procedure consists of ten basic steps which are described below:

1. **Determine Subsystem Peak RII Flow.** In the cost evaluation, RII flow is expressed in terms of a peak flow rate, since the major impact of RII in the sewer system is on the capabilities of facilities to handle peak flows. Typically,

the RII flow will be based on the peak hour (or other suitable short-term) flow for a specified design storm. The choice of design storm conditions may depend on regulatory requirements or simply reflect the degree of conservatism that is desired in sizing facilities.

2. **Estimate RII Distribution.** In a typical subsystem, the RII will not be evenly distributed among all pipes in the area. Certain "worse" pipe reaches, or "minibasins," may have higher unit RII contributions than others, i.e., contribute a greater proportion of the RII flow. In the field, the RII distribution can be determined through flow mapping (flow isolation) or intensive flow monitoring during rainfall.

   Figure F-1 presents a generalized RII distribution envelope. Although the envelope is conceptual in nature, it agrees well with data from several sewer systems in which infiltration (RII or GWI) distributions have been developed based on flow isolation data. Based on a general knowledge of the key RII factors in the subsystem, the envelope can be used to estimate the RII distribution. Typically, older systems with more widespread defects will exhibit a more diffuse distribution (lower envelope boundary), while newer systems might be characterized by more concentrated distributions (upper envelope boundary).

3. **Target the Percentage of the Subsystem for Rehabilitation.** This target value generally represents the point on the RII distribution curve where the curve starts to "level off." Above this percentage, the benefits of rehabilitation, in terms of incremental RII flow removed, begin to decrease. However, the target percentage of the subsystem should at least be large enough to significantly impact the RII flow (e.g., the targeted portion of the subsystem should contribute at least about 50 percent of the RII). For the envelope shown in Figure F-1, the target rehabilitation percentage for a newer system (concentrated distribution) might be about 30 percent of the subsystem, and 50 percent for an older system (more diffuse distribution). In these cases, the amount of the total subsystem RII contributed by the target percentage of the subsystem would be about 80 percent of the RII.

4. **Select the Method of Rehabilitation.** Pipe rehabilitation methods that can be used for RII control were described in Appendix D. Most commonly used methods are grouting and slip-lining. Although in very old, deteriorated systems, it may be necessary to replace a considerable number of pipes or pipe sections because of structural problems, the rehabilitation method selected for the RII cost analysis should be based on rehabilitation for RII correction only. It can be

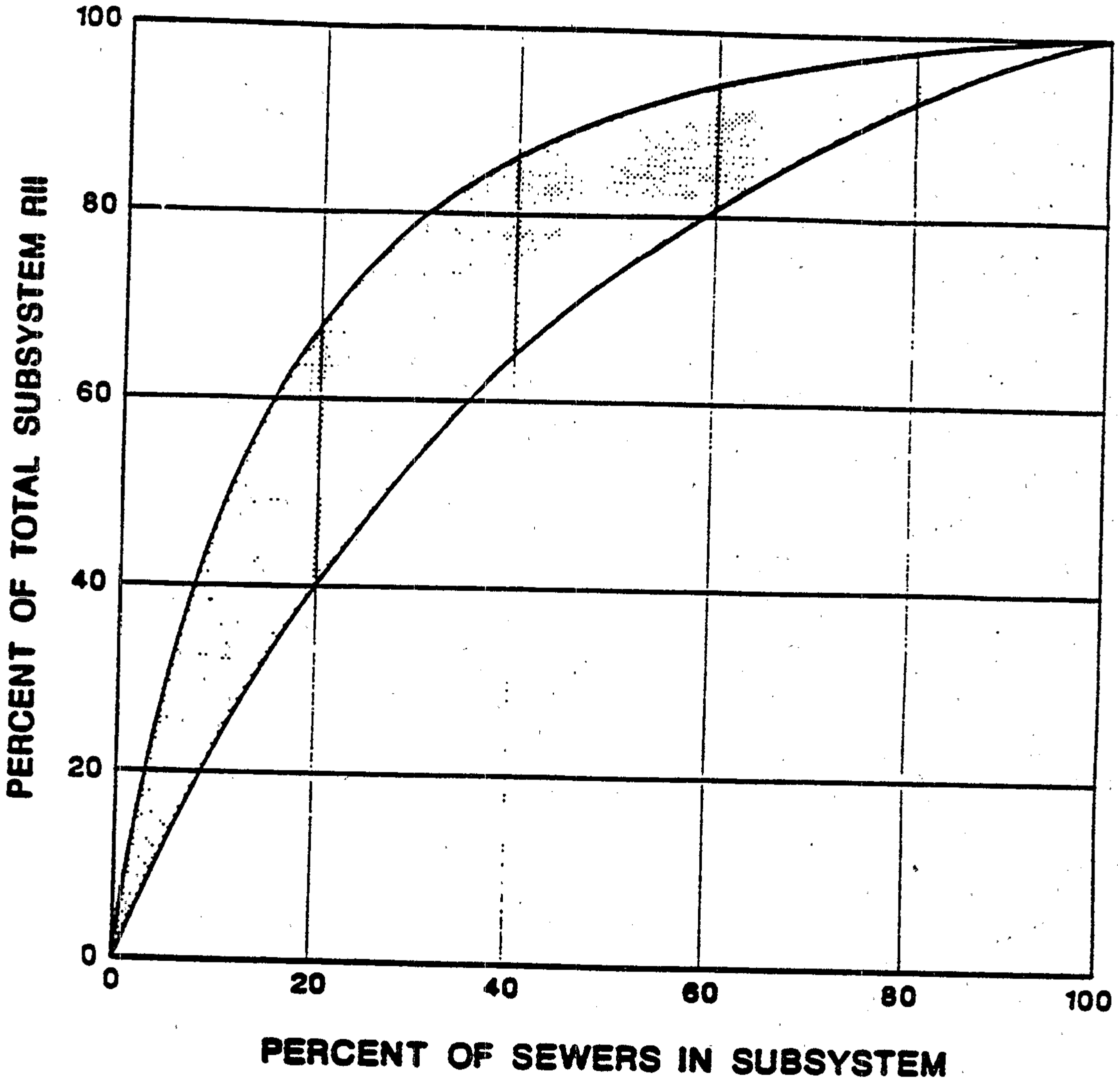


FIGURE F-1

HYPOTHETICAL RII DISTRIBUTION ENVELOPE

assumed that structural repair or replacement would be required regardless of RII considerations.

For this cost analysis, the rehabilitation method is assumed to be grouting or slip-lining. The selection of either method should be based on known conditions in the sewer system. In general, grouting would be most appropriate in newer systems in good structural condition, with few root problems, and in which the groundwater level does not fluctuate below and above the pipes. Slip-lining would be more appropriate for older systems, in areas with extensive root intrusion, or in areas where grout shrinkage could be a problem due to changing soil or groundwater conditions.

5. **Select Rehabilitation Approach.** The rehabilitation approach refers to the extent of rehabilitation in the project area, specifically, whether the project includes only the publicly owned portion of the system, or also addresses the private service laterals. Four rehabilitation approaches are evaluated in this cost analysis:

   - o Isolated repair.
   - o Mains only.
   - o Mains plus the lower portion of service laterals (to property line).
   - o Mains plus entire service laterals (to building).

   In this context, isolated repair could include spot repairs of specific defects or manhole-to-manhole rehabilitation of non-contiguous reaches. The selection of rehabilitation approach may be dictated by financial or institutional constraints

6. **Estimate Rehabilitation Effectiveness.** The significance of the distinction between the four rehabilitation approaches described above is in the amount of RII reduction that can be expected from rehabilitation. In general, the more comprehensive the program, i.e., the more components of the sewer system that are included, the greater reduction that will be achieved. Thus, rehabilitation of the mains plus the lower laterals should achieve a proportionately greater reduction in RII than rehabilitation of the mains only.

   However, because RII will migrate to unrepaired defects, the percentage reduction in RII cannot be directly related to the amount of RII originally contributed by the portion of the system that is rehabilitated. The following estimated ranges for the effectiveness of each of the four rehabilitation approaches are based largely on engineering judgement, but supported by the limited data available from sewer rehabilitation projects for which an assessment of rehabilitation effectiveness has been able to be made.

o	Isolated Repair	0 to 10 %
o	Mains Only	0 to 20 %
o	Mains plus Lower Laterals	30 to 40 %
o	Mains plus Entire Laterals	65 to 80 %

The ranges are intended to reflect different types of sewer systems. For example, the lower end of each range might apply to old sewer systems, and the higher end of the range to newer systems constructed with modern joint materials. For any given system, different assumed reductions might be warranted if such data is available from previous rehabilitation projects, or if known conditions in the system would suggest other values. For example, in the sewer system in North and South Shenango, Pennsylvania, described in Appendix C, a greater rehabilitation effectiveness would be expected through rehabilitation of the mains and lower laterals alone because the mains and lower laterals are known to have defective joints, and the upper portion of the laterals, constructed of different pipe materials, are believed to be relatively watertight.

The rehabilitation benefit percentages presented above are intended to represent reductions in the peak RII flow, rather than the total storm volume of RII. Also, the percentages represent average reductions over the period of the cost analysis (20 years), reflecting the creation of new RII sources due to damage and deterioration of the system over time. Initial reductions would be expected to be higher. For example, a rehabilitation program projected to have an average 70 percent reduction over 20 years might be expected to achieve a 90 percent reduction immediately after construction.

7. **Calculate RII Reduction.** The RII reduction is the rehabilitation benefit percentage (from Step 6) applied to the portion of the total subsystem RII contributed by the rehabilitated portion of the subsystem (from Step 3).

8. **Estimate Rehabilitation Costs.** The cost of rehabilitation depends on the amount of the subsystem included in the rehabilitation program (from Step 3), the selected rehabilitation method and approach (from Steps 4 and 5), and such physical parameters as depth of the sewers, lateral density, and soil and groundwater conditions.

In the cost-effectiveness analysis, the rehabilitation costs are calculated on a present worth basis. Therefore, the useful life of the rehabilitation method must be considered. For example, slip-lining is a relatively "permanent" type of rehabilitation method, and might be considered to have a useful life of about 50 years. Grouting, on the other hand, would typically not last as long because of grout deterioration and development of new RII sources in the previously grouted pipe reach. Therefore, grouting might be assigned a useful life of 5 to 15 years. The determination of useful life might depend on the type of grout to be used, the anticipated quality of the work, the relative age and condition of the sewers, and physical conditions such as groundwater level which may affect the long-term durability of the grout.

9. **Estimate Cost Savings in Transport and Treatment.** The cost savings in transport and treatment is the difference in cost between those facilities required to handle the entire peak flow without system rehabilitation and those required after RII reduction (from Step 7). Transport and treatment costs will be highly dependent on the capacity of existing facilities, as well as the length of trunk sewers and interceptors downstream of the subsystem. Transport and treatment costs must generally be estimated based on the overall plan for the total sewer system, since the incremental cost reductions due to rehabilitation in one single subsystem may not be significant. Therefore, reasonable assumptions must be made regarding potential RII reductions in the other subsystems in the system.

   Although RII correction will reduce the annual operation and maintenance (O&M) costs of the system as well as the capital costs for construction of additional system capacity, the magnitude of the O&M cost savings will generally be very small compared to the capital costs for construction. This is because system facilities must be constructed to carry the design storm peak RII flow, whereas peak flows of this magnitude will occur relatively infrequently. Furthermore, the cost for treatment may not be significantly affected by the peak flows, since treatment schemes will typically be designed for flow equalization or split-stream processing so that costly secondary treatment, for example, is not provided to the entire peak flow (i.e., the plant effluent consists of combined primary and secondary effluent meeting overall plant discharge requirements).

   Whether or not O&M costs are significant will depend both on the treatment plant process and the seasonal rainfall pattern of the area. Since RII is not a sustained flow like GWI and since treatment plants will generally not be designed to process peak hourly flows, the cost to treat the annual volume of RII will generally not be a significant component of total O&M costs.

As with rehabilitation costs, transport and treatment cost savings should be expressed on a present worth basis.

10. **Calculate Cost Effectiveness of RII Control.** The cost effectiveness of RII control is determined by comparing the present worth cost savings in transport and treatment resulting from RII reduction (from Step 9) to the present worth cost of rehabilitation (from Step 8). The ratio of transport and treatment cost savings to rehabilitation cost is termed the "C-E Ratio". A C-E ratio greater than or equal to 1.0 indicates that RII correction is cost effective.

COST EVALUATION OF MODEL SYSTEMS

The cost analysis approach described in the previous section was applied to different "model" sewer systems. The purpose of this exercise was to identify how the cost-effectiveness of RII correction is affected by the characteristics of the sewer system, the type of rehabilitation approach selected, and other variables in the cost calculation.

Model System Descriptions

To facilitate the cost evaluation of model systems using a computer spreadsheet, four basic model system descriptions were developed:

- Type A - Relatively old system generally below the groundwater level.
- Type B - Relatively old system generally above the groundwater level.
- Type C - Relatively new system generally below the groundwater level.
- Type D - Relatively new system generally above the groundwater level.

The designations "old" and "new" are not necessarily intended in the literal sense, but are used to characterize the general construction and condition of the subsystem. Specifically, each subsystem type is intended to describe a particular RII distribution (see Figure F-1), as indicated in Table F-1.

Each of these basic system types were evaluated with respect to several variables, as follows:

- Magnitude of RII flows (as expressed as the ratio of peak RII to average base sanitary flow, ranging from 5 to 20).

- o Density of service laterals (ranging from 10 to 40 per 1,000 feet of sewer main).
- o Rehabilitation approach (isolated repair, mains only, mains plus lower laterals, mains plus entire laterals).
- o Rehabilitation method (grouting or slip-lining).

The cost evaluation was used to identify the relative sensitivity of the cost effectiveness of RII correction to each of these model variables.

Model Assumptions

The following assumptions were used in the cost evaluation:

System Size. The analyzed subsystem was assumed to contain 30,000 feet of sewer main. The subsystem was assumed to be part of an overall sewer system containing 50 similar size subsystems, 20 of which were assumed to have similar RII characteristics and therefore included in the rehabilitation program.

Wastewater Flows before Rehabilitation. Average base wastewater flow (BWF) was assumed to be 70 gpcd. For the analyzed subsystem, average BWF was calculated based on the assumed lateral density in the subsystem, assuming three persons per lateral. For the entire system, average BWF was calculated based on an average of 1,500 persons per subbasin (average lateral density of 16.7 per 1,000 feet). Peak BWF was assumed to be 1.5 times average BWF for the total system flow to the WWTP, and 2.5 times average BWF for a trunk sewer serving the analyzed subsystem and four other similar subsystems. Peak non-rainfall flow was also assumed to include an allowance for "non-excessive" groundwater infiltration (GWI) of 50 gpcd. The peak RII flow in the analyzed subsystem and in the 20 similar subsystems was calculated as the RII/BWF ratio times the average BWF. Peak RII flow in all other subsystems in the system was assumed to be 3 times average BWF. The total peak flow before rehabilitation was calculated as the sum of the peak BWF plus GWI allowance plus peak RII.

Wastewater Flows after Rehabilitation. An assumed effectiveness of rehabilitation (the percentage reduction in the peak RII flow in the rehabilitated portion of the subsystem) was assigned to each system type based on the rehabilitation approach, as indicated in Table F-1. The amount of the RII reduction was then calculated based on the percentage reduction applied to 80 percent of the total subsystem peak RII flow. The same reduction was assumed to occur in the 20 similar subsystems also being rehabilitated. The total peak flow after rehabilitation was calculated as the total peak flow before rehabilitation minus the RII reduction in all rehabilitated subsystems.

Capacity of Existing Facilities. The peak flow capacity of the existing WWTP was assumed to be 2.5 times average dry weather flow. The peak capacities of the interceptor to the WWTP (assumed to carry the entire flow from the system) and the trunk sewer serving the analyzed subsystem (assumed to carry the flow from five similar subsystems) were assumed to be 4 times average dry weather flow.

Cost Basis. The cost analysis was done on a present worth basis assuming a 20-year analysis period and 8-7/8 percent discount rate.

Rehabilitation Costs. Unit costs for grouting and slip-lining were developed as shown in Table F-1. The unit costs were applied to the pipe footage and number of laterals in the rehabilitated portion of the subsystem. The present worth rehabilitation cost was calculated based on a useful life of 50 years for slip-lining (with a salvage value at 20 years based on straight-line depreciation) and a useful life of 5 or 10 years for grouting (with equivalent re-grouting required at the indicated interval).

Transport and Treatment Costs. Based on the capacity of existing facilities and the total peak flows before and after rehabilitation, the additional capacities (for the WWTP, interceptor, and trunk sewer) required before and after rehabilitation were calculated. The cost for additional WWTP capacity was based on providing flow equalization to handle peak wet weather flows in excess of peak dry weather capacity. The costs for additional interceptor
and trunk sewer capacity were based on providing parallel gravity sewers. Unit costs for additional capacity were based on standard cost curves. The length of the interceptor was assumed to be 30,000 feet (about five miles), and the trunk sewer was assumed to be 5,000 feet (about one mile).

The costs of facilities to carry these additional capacities were calculated for the before and after rehabilitation conditions, and the cost savings, or difference between before and after costs, were determined. The cost savings were expressed in terms of present worth values, assuming a useful life of 20 years for WWTP facilities and 100 years for new pipelines. The total cost savings for the WWTP, interceptor, and trunk sewer were distributed equally among all the rehabilitated subsystems (including the analyzed subsystem) served by each facility.

Cost Effectiveness. The cost effectiveness of RII correction (C-E Ratio) was calculated as the ratio of the total cost savings for the analyzed subsystem to the subsystem rehabilitation cost.

Model System Cost Analysis Results

The results of the cost analysis are presented in Table F-2. Based on the assumptions described in the previous section, the general results of the analysis are:

- RII correction is not calculated to be cost effective in subsystem types generally classified as "old."

- RII correction is calculated to be cost effective under certain conditions in subsystem types generally classified as "new," specifically, if peak RII flows are high, lateral density is low, and the mains and entire laterals are rehabilitated.

Under certain very liberal assumptions, grouting was found to be cost effective for isolated repair or mains-only rehabilitation of new systems with relatively high peak RII flows. This was the case only if the rehabilitation effectiveness indicated in Table F-1 could be achieved even if the lateral connections to the main were not included in the rehabilitation program and the useful life of grouting was assumed to be at least 10 years. Since lateral connections are typically significant sources of RII, and since the useful life of grouting depends on a variety of factors, including quality control during construction, these may not be realistic assumptions. In general, assuming a five-year versus a ten-year useful life for grouting reduces the cost effectiveness of RII correction by 40 to 50 percent.

TABLE F-1

MODEL SYSTEM COST ANALYSIS ASSUMPTIONS

Subsystem Type	Description	Percent of Subsystem Rehabilitated[a]	Rehabilitation Effectiveness (Percent RII Reduction): Isolated Repair	Mains Only	Mains + Low. Lat.	Mains + Entire Lat.
A	Old, Below Groundwater	60	0	10	30	70
B	Old, Above Groundwater	50	0	10	30	70
C	New, Below Groundwater	40	10	20	40	80
D	New, Above Groundwater	30	10	20	40	80

		Unit Cost ($)[b]: Isolated Repair[c]	Mains Only	Mains + Low. Lat.	Mains + Entire Lat.
Rehabilitation Unit Costs					
Mains (per lf)	Slip-lining	8.00	24.00	24.00	24.00
	Grouting	2.50	7.25	7.25	7.25
Laterals (per lateral)	Slip lining	130[d]	400[d]	1500	2600
	Grouting	0	0/400[d]	900	1500

[a] Percent of subsystem contributing 80 percent of total subsystem RII

[b] Construction cost.

[c] Based on approximately one third of rehabilitation area

[d] Cost to excavate and reconnect lateral or rehabilitate lateral connection

TABLE F-2

COST ANALYSIS RESULTS

Subsystem Type[a]	Lateral Density (lat./1,000 ft)	RII/BWF Ratio	C-E Ratio: Isolated Repair[b,d]	C-E Ratio: Mains Only	C-E Ratio: Mains & Low Lat.	C-E Ratio: Mains & Entire Lat.	Rehabilitation Method
A	10	10	0	0	0.1	0.1	Slip-lining
A	10	20	0	0.1	0.2	0.5	Slip-lining
A	40	10	0	0.1	0.1	0.2	Slip-lining
A	40	20	0	0.1	0.2	0.3	Slip-lining
B	10	10	0	0	0.1	0.1	Slip-lining
B	10	20	0	0.1	0.2	0.6	Slip-lining
B	40	10	0	0.1	0.2	0.3	Slip-lining
B	40	20	0	0.1	0.2	0.4	Slip-lining
C	10	10	0.3	0.1/0.2[c]	0.2	0.5	Grouting[d]
C	10	20	0.6	0.4/0.7[c]	0.6	1.2	Grouting[d]
C	40	10	1.6	0.3/0.9[c]	0.3	0.5	Grouting[d]
C	40	20	1.1	0.4/1.4[c]	0.5	0.7	Grouting[d]
D	10	10	0.4	0.1	0.2	0.5	Slip-lining
D	10	20	0.8	0.4	0.5	1.1	Slip-lining
D	40	10	2.1	0.3	0.3	1.1	Slip-lining
D	40	20	1.4	0.5	0.5	0.7	Slip-lining

a See Table F-1 for subsystem type description and assumptions.
b Grouting only for isolated repair.
c With/without repair of lateral connections.
d Assumed 10 year useful life for grouting